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FOREWORD

It is past two hundred years that India is mining Coal. The recorded history of mining metal dates back to Mourya Dynasty. The legislation for mining was laid down by Kawtillya. The Indian miners in old days have opened and worked mines in Africa.

Even with the historic past of mining heritage we have very few books on mining and allied subjects to the credit of Indian authors.

The book "A Guide to Mine Surveying" by Sri Sukumar Ghatak is a well taken step by a teacher for fulfilment of the need of students and Surveying & Mining Engineers in the field.

It has given the pleasure to read that this treatise covers all branches of Surveying (Coal & Metal) and preparation & maintenance of plans All along the approach to the subject matter, every care has been taken to arrange matter from simpler to harder with full details including illustrations. A large number of graded worked out examples have been set in systematic manner and logical sequence, to understand the subject much better The treatment of the subject has been accurate, clear and given wide coverage
This book will be an asset to Library. Students and Engineers.

N. R. MITRA
A.I.S.M., P.G. DIP M1N (SHEFFIELD), F.C.C.
Formerly,
Chairman - Cum - Managing Director
Bharat Coking Coals Limited.

PREFACE TO THE FIRST EDITION

A Guide to Mine Surveying, in three volumes, is the first Guide-Cum-Text book in Mine Surveying written in the form of Questions and Answers abound with suitable illustrations. While preparing these volumes, I have always kept in mind the requirements of students regarding the latest as well as the changing trend of the examinations. Answers to the questions selected from papers set in the various examinations of recent date have been systematically arranged chaptcrwisc, and as the working of the problems in Mine Surveying is essential to grasp the subject matter easily, a large number of typical numerical problems, covering all important types, have been included at the end of each chapter. Chapters on exercises have been added for the students to solve them independently The students will find them of immense help from the point of view of success in the examinations.

This volume, containing the chapters on Mine plans, scales and mine models, Chain surveying, Dial surveying, Levelling, and plane table surveying, is specially written for the students preparing for eompetetive examinations both under the Coal and Metalliferous Mines Regulations. Students will find this volume most useful and indispensable to them.

I gladly express my indebtness and deep gratitude to Shri N. R. Mitra, Director, Eastern Coalfields Ltd, for his valuable suggestions and encouragement which had made me possible to produce the book in present form. I also like to express my thanks to Shri S. K. Choudhury, Regional Controller of Mines, I.R.M., for his valuable suggestions, and to Shri Sudhir Kumar Ghatak, Editor, Coalfield Lxprcss, for the personal interest he took in printing and keeping vigilant eye on the quality of the book.

Although every care has been taken to avoid mistakes and misprints, it is difficult to claim perfection. Some errors may have crept in, and I shall be obliged to those readers who will point out them. Constructive criticisms and suggestions from Students, Practising Mining Engineers and Mine Surveyors will also be welcome.

UKHRA, BURDWAN
15th July, 1980

S. GHATAK, W.B.G.S.

PREFACE TO THE SECOND EDITION

The great success *of A GUIDE TO MINE SURVEYING* in three volumes has encouraged me to produce the volumes in standard forms by re-arranging the chapters and including many new techniques, questions-answers, worked out examples, exercises with answers. The title of the volumes has also been changed to *A GUIDE TO MINE SURVEYING AND LEVELLING,*.

In this edition, the volume one contains eight chapters. There is sufficient material in each chapter and the work has been arranged so that a student may derive the greatest bcnifit by studying each chapter in its order. "Worked examples" are included in numerous places, whilst at the end of most chapters is a list of exercises with numerical answers after the question. All these have been selected from question papers set by the appropriate examining bodies.

Suggestions to improve the work will be greatly appreciated.

Asansol, the 1 st May, 1986 **S. GHATAK**

PREFACE TO THE THIRD EDITION

A growing professional awareness, .the trend of mining legislation in India and the progress made in the design and precision of surveying instruments and equipment have combined to raise the standard and increase the scope of mine surveying. Students have felt the need for such book as are designed to Cope with the present trend and help them to higher posts in mining industry.

The increasing popularity of this book has encouraged me to come out with this third edition, thoroughly revised and extensively enlarged with the inclusion of a new chapter and plenty of new material. It is hoped that this edition should prove extremely useful .to the students.

Constructive criticisms, suggestions and corrections to improve the work will be highly welcome.

Asansol, the 24th January, 1990 **S. GHATAK**

PREFACE TO THE SIXTH EDITION

I deem it a great privilege in presenting to my ever-increasing number of esteemed readers, a thoroughly overhauled and somewhat enlarged sixth edition of this popular book which was introduced in 1980 This volume has nou been revised supplemented and updated with fresh materials. A new chapter (Objective Type Questions with Answers) has been included in this edition so as to keep pace with the changing trend of the examinations

S. GHATAK

SKYLARK
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24th January, 2002

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CHAPTER - I

INTRODUCTION

★★★

between the arc and the subtended chord on the surface of earth is only 0.1 metre in 18.2 kilometres. The degree of accuracy required in plane surveying is comparatively low. Approximately areas less than 260 sq. km are considered as plane. Therefore, plane surveying is quite justifiable for all ordinary mining and engineering surveys.

2. GEODETIC SURVEYING : The survey in which the effect of curvature of earth is taken into account is called Geodetic Surveying.

The earth is an oblate spheroid of revolution, the length of its polar axis (12,713.800 metres) being somewhat less than of its equatorial axis (12,756.750 metres). Thus, the polar axis is shorter than the equatorial axis by 42.95 kilometres. Relative to the diameter of the earth this is less than 0.34 percent.

If we neglect the irregularities of the earth, the surface of the earth is a curved surface. When the area to be surveyed is large, this curvature effect of earth can not be ignored. All survey lines lying on the earth's surface are therefore curved lines and the triangles are spherical triangles. All Geodetic Surveys, therefore, involves spherical trigonometry.

The object of geodetic survey is to determine the precise positions on the surface of the earth, of a system of widely distant points which form control stations to which surveys may be referred. Geodetic surveys are characterized by the large distances and areas, the use of very refined instruments, the use of very refined methods of observations and adjustments and high precision. Such surveys are conducted only through the agencies of Governments.

CLASSIFICATION . Surveys may be classfied under headings which define the uses of the resulting maps :

1. Classification according to the object of survey -

(i) **Geological Survey :** The survey which is conducted for determining different strata in the earth's crust is known as Geological Survey.

(ii) **Mine Survey :** The survey which is done for exploring mineral wealth under the earth's crust, such as coal, copper, gold, etc. is known as Mine survey.

(iii) **Military Survey :** The Survey which is conducted for determining the best positions of attack and defence from military point of view, and also for finding out the best storage size for keeping ammunition is known as Military Survey.

(iv) **Archaeological Survey :** The survey which is done to trace out the relices of the past is known as Archaeological Survey.

(v) **Engineering Survey :** The survey which is conducted to collect data for carrying out any engineering project such as construction of road, railway, dam, water supply, etc. is known as Engineering Survey.

2. Classification according to the nature of the field of survey -

(i) **Land Survey :** The survey which is carried out for determining the relative positions of objects on the earth's surface is called land survey. This may be further sub-divided

into the following classes -

(a) Topographical Surveys for determining the natural features of a country such as hills, valleys, rivers, lakes, etc. and also, the artificial features such as roads, railways, canals, buildings, towns, villages, etc.

(b) Cadastral Surveys in whcih additional details such as boundaries of fields, houses, and other land properties are determined.

(c) City Surveys for laying out plots, streets and for constructing water supply systems, sewers and other works.

(ii) Hydrographic or Marine Survey : It deals with bodies of water for purpose of nevigation, water supply, harbour works or for the determination of the mean sea level.

(iii) Astronomical Survey : It offers the surveyors means of determining the absolute location of any point or the absolute location and direction of any line on the surface of the earth This consists in observations to the heavenly bodies such as the Sun or any fixed star.

3. Classification according to the methods employed in survey -

(i) Triangulation Survey : The system of surveying in which all the survey stations selected in the area are connected with survey lines in such a way that they form net-work of triangles is called triangulation survey.

(ii) Traverse Survey : The system of surveying in which whole of the area to be surveyed is divided into various traverse with any figure formed by four or more lines is called traverse survey.

4. Classification according to the instrument used -

An alternative classification may be based upon the various instruments used for the survey, the chief types are -

(a) Chain-Survey.
(b) Compass Survey.
(c) Levelling
(d) Plane Table Survey.
(e) Theodolite Survey.
(f) Tacheometric Survey.
(g) Photographic Survey.
(h) Aerial Survey.
(i) EDM Survey.

BASIC PRINCIPLES OF SURVEYING

The fundamental principles upon which the various methods of plane surveying are

based can be stated under the following two aspects :

1. Location of a point by measurement from two points of reference .

The relative positions of the points to be surveyed should be located by measurment from at least two points of reference, the positions of which have already been fixed. This may be done by (a) linear measurements, (b) angular measurments, or (c) both linar and angular measurements.

2. Working from whole to the part .

Under this principle, whether plane or geodetic surveying, it is very essential to establish first a system of control points with higher precision of the whole area to be surveyed. The whole area will then be divided into large triangles or large traver-es or both. They are then further sub-divided into smaller one's by locating other control points in between the main control points with less precise methods. The details can be located afterwards. In this way the minor errors are automatically controlled and localised. They do not accumulate.

On the other hand if work from part to the Whole, small errors will increase in the process of expansion of survey and will become uncontrollable at the end.

A good surveyor should always bear the following principles in mind for conducting surveys :

1. Always work from whole to the part.
2. Always choose the method of survey that is the most suitable for the purpose.
3. Always make provisions of adequate checks.
4. Always record field data carefully.

MEASUREMENTS

Measurements are of two types :

(a) Linear measurements.

(b) Angular measurements.

(a) Linear measurements : These are further sub-divided into -

(i) Horizontal distances : In surveying all measurements of distances are horizontal. Sometimes however inclined measurements are made for convenience as in undergroud roadways, but they must be subsequently reduced to their horizontal equivalents for plotting purposes.

(ii) Vertical distances : The measurements taken in the vertical plane are called vertical distances.

(b) Angular measurements : These are also further sub-divided into -

(i) Horizontal Angles : The angular measurements taken in the horizontal plane are called horizontlal angles.

(ii) Vertical Angles : The angular measurements taken in the vertical plane are called vertical angles.

Instruments used for taking measurements :

Following are the instuments used for taking different types of measurements in survey -

1. Chain or tape is used for taking direct linear measurements.
2. Tacheomenter and subtense bar are used for taking direct and indirect linear measurements.
3. Prismatic compass, miner's dial, theodolite, box-sextant, etc. are used for taking angular measurements.
4. Abney level, tangent clinometer etc. are used for finding grades, laying grades and for finding heights.
5. Theodolites, levels, etc. are used for finding difference of levels (vertical distances) between different points.
6. Plane table is used for surveying and plotting simultanously in the field.
7. Photo-theodolite is used for photographic survey.
8. Modern E.D.M. instruments are used for distance measurements.

Total Station System, an electronic tacheometer measures the horizontal and vertical angles and distances electronically saving time of observations and calculations. It is also capable of calculating the planimetric positions and levels instantaneously. An "electronic field book" can be interfaced with the instrument to record and store not only the angles and distances but also the co-ordinates and levels of the points and other related informations. The least count is 1mm for lenear measurements and 1" or less for angular measurements. Total Station System is highly recommended for subsidence measurements.

UNITS OF MEASUREMENTS : There are two systems of measurements -

1. Metric System or M. K. S. (Metre, kilogram, Second)
2. British System or F. P. S. (Foot, Pound, Second)

Linear Measurements : The unit of linear measurements is metre in metric system. An international Prototype Metre, made of 90% platinum and 10% iridium alloy, kept in France had been used for standardisation purpose.

BASIC UNITS

British Units		Metric Units	
Length		*Length*	
12 inches	= 1 foot	10mm	= 1 centemetre
3 feet	= 1 yard	10cm	= 1 decimetre
1760 yards	= 1 mile	10dm	= 1 metre

British Units	Metric Units
Length 6 feet = 1 fathom 6080 feet = 1 nautical mile	*Length* 10 m = 1 decametre 10decam = 1 hectametre 10hm = 1 kilometre
Area 144 sq.Inch = 1 sq.foot 9sq.feet = 1 sq.yard 43560 sq. feet = 1 acre 4840 sq.yards = 1 acre 45 sq. feet = 1 chattak 16 chattak = 1 kattah 20 kattah = 1 bigha	*Area* 100 sq.m = 1 are 100 ares = 1 hectare 100 hectares = 1 sq.km
Volume 1728 cu.inches = 1 cu foot 27 cu.feet = 1 cu yard	*Volume* 1000 cu.mm = 1 cu.cm 1000 cu. cm = 1 cu.decim 1000 cu. decim = 1 cu.m

CONVERSION FACTOR

1 metre	= 1.0936	yards	= 3.2808	feet	= 39.37 inches
1 km	= 0.6214	mile		1 mile	= 1.6093 km
1 inch	= 2.54	cm		1 foot	= 0.3048 m
1 sq.m	= 10.7639	sq.ft		1 sq.ft	= 0.0929 sq.m
1 sq.inch	= 6.4516	sq.cm		1 acre	= 0.4047 hectare
1 cm.inch	= 16.387	cu.cm		1 cu.ft	= 0.028 cu.m
1 cu.yard	= 0.7645	cu.m		1 cu.m	= 1.308 cu.yard
1 gallon	= 4.5456	litres		1 gallon	= 4 quart
1 kg	= 2.2046	lbs		1 tonne	= 0.9843 ton
1 cu.m water = 1000		litres = 1000kg or		1 tonne by weight	

Angular Measurements : The radian is the unit of plane angle (π radians = 180°). Three

popular systems of angular measurments are -

(a) *Sexagesimal System*			(b) *Centesimal System*		
1 circumference	= 360°	(degrees)	1 circumference	= 400^g	(grad)
1 degree	= 60'	(minutes)	1 grad	= 100^c	(centigrad)
1 minute	= 60"	(seconds)	1 centigrad	= 100^{cc}	(centi-centigrad)

(c) *Hours Seystem*

1 circumfernce	= 24^h	(hours of time)
1 hour	= 60^m	(minutes)
1 minute	= 60^s	(seconds)

The Sexagesimal System is widely used in surveying instruments. The Centesimal System used in Europe offers facility in computation and interpolation. The Hour System is mostly used in astronomy and navigation.

DIFFERENCE BETWEEN A PLAN AND A MAP

PLAN : A plan is the graphical representation of the features on or below the earth's surface as projected on a horizontal plane of the drawing sheet. As a plan represents the area on a horizontal plane, the horizontal distances are measured between various points on the earth's surface or belowground. The solpe distances are always reduced to equivalent horizontal distances. A plan is drawn on a relatively large scale.

MAP : If the scale of the graphical projection on a horizontal plane is small, the plan is termed as map. In addition, a map generally shows some addtional features, such as reliefs, hachures and contour lines to indicate undulations on the ground.

SCALE OF A PLAN AND A MAP : The term "scale" indicates the relationship between the length of a unit distance on the map or plan and the actual distance which that unit represents on the field. Scale can be represented by the following method :

(1) *Engineer's Scale :* One cm on the plan represents some whole number of metres on the ground, e.g. 1cm = 10m.

(2) *Representative Fraction (R.F.) :* One unit of length on the plan represents some number of same units on ground. For example, if the scale is 1cm = 10m

$$R.F = \frac{1}{10 \times 100} = \frac{1}{1000} \quad \text{or} \quad 1:1000$$

The above two types of scales are also known as *numerical scales.*

(3) *Graphical Scale :* A graphical scale is a line sub-divided into plan distance corresponding

to convenient units of length on the ground.

If the plan or map is to be used after few years, the numerical scales may not give accurate results due to shrinkage of the sheet or paper. However, if a graphical scale is also drawn, it will shrink proportionately and the distances can be found accurately. That is why, every mine plan shall show a scale of the plan at least 25cm long and suitably sub-divided.

The scale of a map varies according to its purpose, size and the required precision of plotting. Since most maps show all the surface features which can be readily drawn on them it follows that the smaller the scale the detail can be shwon, but greater will be the surface area covered.

Scales are generally classified as large, medium and small as under :

Large scale	:	1 cm	= 10m or less than 10m
Medium scale	:	1 cm	= 10m to 100m
Small scale	:	1 cm	= 100m or more than 100m

CHAPTER - II

MINE SURVEYING AND LEGISLATION

★★★

which often involve comparative short traverse lines and greater susceptibility to error. Under such conditions the mine surveyor can and does achieve a surprisingly high standard of work with intelligence and confidence.

Appointment of Surveyors : (1) At every mine one or more persons not less than 23 years of age and holding a Surveyor's Certificates shall be appointed to be the Surveyor for carrying out the surveys and levelling and for preparing the plans and sections required under the Act or the regulations or orders made thereunder.

(2) No person shall be appointed as a surveyor of more than one mine or in any other capacity in the same mine, without the previous permission in writing of the Regional Inspectors and subject to such conditions as he may specify therein, Such permission may be granted only when the average output of the mine does not exceed 2,500 tonnes. The Regional Inspector may, by an order in writing, revoke such permission at any time.

(3) The number of surveyors required to be appointed shall be on the following scale :

Average monthly output in tonnes	Number of Surveyors required
Below 8,000	One
Above 8,000	One for every additional 15,000 tonnes or part thereof.

For calculating the above output, only half the output obtained from depillaring operations or from the opencast mines shall be taken into consideration.

The Regional Inspector may, by an order in writing, and subject to such conditions as he may specify therein, permit or require the appointment of surveyors in variation of these provisions.

(4) If a mine has more than one surveyor, each shall carry on the duties and responsibilities of the surveyor for the part or section of the mine to be assigned in writing by the owner, agent or manager.

The owner, agent or manager shall appoint one of the surveyors to be responsible for the preparation and maintencance of the plans required to be prepared and maintained under the regulations who also be responsible for the co-ordination and overall supervision of survey work in the mine.

If a surveyor is assigned duties by owner, agent or manager in respect of more than one mine, it is obligatory on the part of the surveyor to obtain prior permission of the Regional Inspector of mines. Where more than one surveyors are appointed each assigned with duties relating to one mine and only one of them is authorised under the regulation, the surveyor who prepares a plan, is responsible for its accuracy and must sign the plan, section and tracing by him. Similarly, when a plan of constituted mine is prepared after correlation survey of different units, the surveyor who makes such survey and prepares or supervises the preparation of the correlated plan, shall be responsible for its accuracy and shall sign the plan.

Duties and Responsibilities of Surveyors :

1. The surveyor shall -

(a) make such accurate surveys and levellings and prepare such plans and sections and tracings thereof, as the manager may direct or as may be required by the Act or by the regulations or orders made thereunder, and shall sign the plans, sections and tracings and date his signature; and

(b) be responsible for the accuracy of any plan and section, or tracing thereof that has been prepared and signed by him.

2. The surveyor shall record in a bound-paged book kept for the purpose -

(a) the full facts when workings of the mine have approached to about 75 metres from the mine boundary or from disused or water-logged workings;

(b) any doubts which may exist concerning the accuracy of the plans and sections prepared under these regulations; and

(c) any other matter relating to the preparation of the plans and sections that he may like to bring to the notice of the manager.

Every entry in the book shall be signed and dated by the surveyor and countersigned and dated by the manager.

Where in any mine two or more surveyors are employed, each of the surveyors shall make the entries aforesaid in respect of the workings in the jurisdiction or of the plans and sections in his charge.

However, the mine management shall not be absolved of their responsibility under the Act, regulations, etc. to ensure that every plan or section prepared, kept, or submitted under the statutory provisions is (i) correctly maintained, and (ii) updated as required thereunder.

A newly appointed surveyor should pay special attention to the following while scrutinising old plans and survey records of the mine having extensive old workings :

(1) All old plans should be regarded as indicating, not the position of old workings, but merely that workings existed somewhere in the vicinity. The safest procedure is never to take the position of old workings for granted until they have been proved by boring or otherwise.

(2) Accuracies of the plans must be verified. The inaccuracies of the old plans may include (a) Omission to show the full extent of old workings, (b) Omission to correlate the workings with surface, (c) Shrinkage, deterioration, and deprecit condition of the plans, (d) Omission to check and record of the variation of the magnetic meridian and apply such variations in plotting latter workings.

(3) Outlines of all abandoned workings and depth of such workings.

(4) Faults, dykes, etc. in relation to the workings.

(5) Extent of old workings. In the case of thick seams, horizon of the working need to be examined with the help of spot levels and stratum contours. The extent of open ended galleries may be different from those indicated on plans.

(6) The nature and thickness of surrounding strata, also the sources of surface water should be carefully emamined.

(7) Thickness of barriers against waterlogged workings.

(8) Whether plans are plotted by co-ordicate systems and records kept for the same.

(9) Surface drainage system of the mine and the highest flood level of the area.

(10) Workings in the proximity of bodies of water on the surface or underground (a)

location of bodies of water, (b) water in old workings, (c) position of old workings, (d) warning lines.

All important plans and sections including the water danger plan and abandoned mine plans should be thoroughly studied and the records in connection with the workings and plans should be scrutinised.

If any doubt arises as to the accuracy of the plans and sections the surveyor shall have accurate plans and sections prepared afresh before any drivage or other work of development or depillaring.

Charge-Report of Surveyors : Where an outgoing surveyor of a mine hands over charge to an incoming surveyor, he should, in his charge report, point out the incoming surveyor the following -

1. A general information of the mine.
2. An introduction to the survey office.
3. An introduction to the plans, sections and records.
4. A list of instruments and accessories mentioning the instruments which require overhaul and repair.
5. Individual service records of the staff members under him.
6. Important survey works under execution requiring special attention to the surveyor.
7. Sources of danger associated with the mine.
8. Other important matters from the point of view of safety required to be attended by the incoming surveyor.

PLANS AND SECTIONS

General requirements about mine plans - (1) Every plan or section prepared or submitted in accordance with the provisions of the regulations shall -

(a) show the name of the mine and of the owner, and the purpose for which the plan is prepared.
(b) show the true north, or the magnetic meridian and the date of the latter;
(c) show a scale of the plan at least 25 centimetres long and suitably sub-divided;
(d) unless othrwise provided, be on a scale having a representative factor of 1200 : 1.
(e) provided that the Chief Inspector may, by an order in writing and subject to such conditions as he may specify therein, permit or require the plans to be prepared on any other suitable scale; and
(f) be properly inked in on durable paper or on tracing cloth, and be kept in good condition.

(2) The conventions shown in the Second Schedule shall be used in preparing all plans

and sections required by the regulations.

(3) The plans and sections shall be accurate and maintained and corrected up-to-date which is not earliar than three months :

Provided that where any mine or seam or section is proposed to be abandoned or the working thereof to be discontinued or rendered inaccessible, the plan and section shall be brought up-to-date before such abandonment or at the time of discontinuance, as the case may be, such abandonment or discontinuance has been caused by circumstances beyond the control of the owner, agent or manager, in which case the fact that the plan or section is not up-to-date shall be recorded on it.

(4) Plans and sections required to be maintained under the regulations shall be kept available for inspection in the office at the mine, and shall not be removed therefrom, except by or with the approval in writing of the Regional Inspector, unless a true copy thereof has been kept therein.

Types of plans - (1) The owner, agent or manager of every mine shall keep the following plans and sections.

(a) A surface plan showing every surface feature within the boundaries, such as telephone, telegraph or power transmission line, watermain, tram line, railway, road, river, watercourse, reservoir, tank, bore-hole, shaft and incline opening, open-cast working, subsidence and building on the surface.

(b) An under ground plan showing -
   (i) the positions of the workings of the mine below ground;
   (ii) every bore-hole and shaft with depth, incline opening, cross measure drift, goaf, fire stopping, or seal, water dam (with dimensions and other particulars of construction), pumping station and haulage roadway;
   (iii) every important surface feature within the boundaries, such as railway, road, river, stream, water-course, tanks, reservoir, open-cast working and build ing which is within 200 metres of any part of the working measured on the horizontal plane;
   (iv) the general direction and rate of dip of the strata;
   (v) such sections of the seam as may be necessary to show any substantial variation in the thickness or character thereof and showing the working section, and such sections of the strata sunk or driven through in the mine or proved by boring as may be available;
   (vi) the position of every roll, wash-out, dyke and every fault with the amount and directions of its throw; and
   (vii) an abstract of all statutory restrictions in respect of any specified workings with a reference to the order imposing the same.

Whenever this plan is brought up-to-date, then positions of the workings shall be shown by a dotted line drawn through the ends of the workings, and such dotted line shall be marked

and sections required by the regulations.

(3) The plans and sections shall be accurate and maintained and corrected up-to-date which is not earliar than three months :

Provided that where any mine or seam or section is proposed to be abandoned or the working thereof to be discontinued or rendered inaccessible, the plan and section shall be brought up-to-date before such abandonment or at the time of discontinuance, as the case may be, such abandonment or discontinuance has been caused by circumstances beyond the control of the owner, agent or manager, in which case the fact that the plan or section is not up-to-date shall be recorded on it.

(4) Plans and sections required to be maintained under the regulations shall be kept available for inspection in the office at the mine, and shall not be removed therefrom, except by or with the approval in writing of the Regional Inspector, unless a true copy thereof has been kept therein.

Types of plans - (1) The owner, agent or manager of every mine shall keep the following plans and sections.

(a) A surface plan showing every surface feature within the boundaries, such as telephone, telegraph or power transmission line, watermain, tram line, railway, road, river, watercourse, reservoir, tank, bore-hole, shaft and incline opening, open-cast working, subsidence and building on the surface.

(b) An under ground plan showing -
   (i) the positions of the workings of the mine below ground;
   (ii) every bore-hole and shaft with depth, incline opening, cross measure drift, goaf, fire stopping, or seal, water dam (with dimensions and other particulars of construction), pumping station and haulage roadway;
   (iii) every important surface feature within the boundaries, such as railway, road, river, stream, water-course, tanks, reservoir, open-cast working and build ing which is within 200 metres of any part of the working measured on the horizontal plane;
   (iv) the general direction and rate of dip of the strata;
   (v) such sections of the seam as may be necessary to show any substantial variation in the thickness or character thereof and showing the working section, and such sections of the strata sunk or driven through in the mine or proved by boring as may be available;
   (vi) the position of every roll, wash-out, dyke and every fault with the amount and directions of its throw; and
   (vii) an abstract of all statutory restrictions in respect of any specified workings with a reference to the order imposing the same.

Whenever this plan is brought up-to-date, then positions of the workings shall be shown by a dotted line drawn through the ends of the workings, and such dotted line shall be marked

with the date of the last survey.

(c) Where a seam has an average inclination of more than 30 degress from the horizontal, one or more vertical mine section or sections, as may be required by the Regional Inspector, showing a vertical projection of mine working.

(d) A ventilation plan and section where necessary, showing the system of ventilation in the mine, and in partiular -

(i) the general direction of air-current;

(ii) every point where the quantity of air is measured;

(iii) every air-crossing, ventilation door, stopping and every other principal device for the regulation and distribution of air;

(iv) every fire stopping and its serial number;

(v) every room used for storing inflammable material;

(vi) the position of fire-fighting equipments;

(vii) every water-dam with dimensions and other particulars of construction;

(viii) every pumping, telephone and ambulance station; and

(ix) every halulage and travelling roadways.

(e) A geological plan of the area of leasehold, on a suitable scale.

(2) Separate plan and sections for the workings of every seam or of every separate section of every seam shall be kept in respect of clauses (b), (c) and (d) above .

Provided that in respect of underground plans, combined plans of all seams which are lying within 9 metres of each other and which are worked at the mine shall also be kept; and in the combined plans, working in different seams or sections shall be shown in different colours.

(3) (a) The surface and underground plans shall also show surface contour lines drawn at vertical intervals not exceeding 5 metres.

(b) The underground plans shall also show spot levels on the floor of the workings -

(i) along haulage roadways at every roadway junction, except in roadways where tramming is done by manual means where the spot levels may be shown at points not more than 150 metres apart; and

(ii) in the case of headings which have been discontinued either temporarily or premanently, also at the end of such headings.

Where two drifts in stone or galleries in coal pass over one another, this shall be clearly indicated on the plans with appropriate noting, if necessary.

(c) A permanent bench-mark shall be established in the surface and all levels taken above and below-ground shall be referred to a plan in relation to such bench-mark. Particulars of the bench-mark, together with its height above Mean Sea Level, shall be shown on the plans required to be maintained under these regulations.

(4) (a) (i) The surface and underground plans shall also show the settled boundary of the mine, or where the boundary is in dispute, the boundaries claimed by the owner of the mine and by owners of the mines adjacent to the disputed boundary.

Provided that where it is not possible to show the complete boundary of leasehold on the same plan, additional key plan on any other suitable scale showing such boundaries and

the outline of the working shall also be maintained.

(ii) The underground plans required to be kept under the regulation shall also show the workings, and all features, both above and belowground of all adjacent mines as are situated within 60 metres, measured on any plane, of the boundary claimed by the owners of the mine.

(b) The owner, agent and manager of every mine shall as soon as its workings extend to within 60 metres of the settled boundary with an adjancent mine (or where the boundary is in dispute within 60 metres of the boundary claimed by the owner of the adjacent mine) inform the owner, agent or manager of such mine of the fact of such extension and shall also give all reasonable facilities to the surveyors of its adjacent mines to carry out the surveys and levellings required to be made.

(5) (a) The Regional Inspector may, by an order in writing, require such additional details to be shown on the plans and sections required to be kept under the regulations, or the preparation and maintenance of such other plans and sections showing such detail and on such scale within such times as he may specify in the order.

(b) Teh Regional Inspector may, by an order in writing, require the owner, agent or manager to submit to him within such time such plans and sections, or tracings thereof, as he may specify in the order.

(c) The owner, agent or manager shall, at any time, if required by the Regional Inspector, show on any plan or section the then position of the workings of the mine.

Copies of the plans and sections to be submitted - The owner, agent or manager shall on or before the 31st October of every year, submit to the Chief Inspector two up-to-date copies of the plans and sections, The provisions of the regulations shall be deemed to have been complied with if the owner, agent or manager gets the copies of plans and sections submitted hereunder during the previous year brought up-to-date at his own expense.

Plans and sections to be submitted after abandonment or discontinuance -

(1) Where any mine or seam or section thereof is abandoned or the working thereof has been discontinued over a period exceeding 60 days, the person who was the owner of the mine at the time of the abandonment or discontinuance shall, within 30 days after the abandonment or within 90 days after the discontinuance of the working as the case may be, submit to the Chief Inspector two true copies of the up-to-date plan and section of the workings of the mine or seam or section maintained under the Regulations. Every such copy shall show the bearing and distance of at least one of the shafts or opening of the mine from a tri-junction or revenue pillar or from any other prominent and permanent surface feature, the position of all water dams built belowground (with their dimensions and other particulars of construction) and also the spot levels at the end of the workings :

Provided that if a change of ownership occurs after the abandonment or discontinuance and before the expiry of the 30 days or the 90 days aforesaid, as the case may be, such plans and sections shall be submitted forthwith.

(2) The original or a certified true copy of the plan and section submitted shall be kept in the office at the mine.

(3) The Chief Inspector may, on such conditions as he thinks fit to impose, and on payment of the cost of preparing copies as determined by him, supply copies of a plan or section submitted to him under the regulation (1) or such parts thereof as he thinks fit -

(a) to any person having a bonafide interest in the mine, seams or section; or

(b) to the owner, agent or manager of an adjacent mine.

Survey instruments and materials - The owner or agent shall provide acurate and reliable survey instruments and materials for the proper carrying out of all survey and levelling work and for the preparation of the plans and sections required under the regulations; and no other instruments shall be used in connection with any such survey or levelling work.

List of plans, sections and instruments and their storage - (i) All plans and sections, and tracings or copies thereof, kept at the mine shall be serially numbered.

(2) Suitable arrangements shall be made at every mine for the proper storage and maintenance of every plan and section and of all instruments and materials. Such arrangements shall provide for flat storage of every underground plan and vertical section.

(3) Every field book and other notes used in the preparation of plans and sections required under the regulations shall be duly indexed and kept in the office at the mine.

(4) A list of all plans and sections maintained under the regulations, or any orders made thereunder and tracings or copies thereof, of all survey instruments provided under the regulation with their respective types, specifications and identification numbers and of all field books and other notes shall be kept in a bound-paged book kept for the purpose, and shall be brought up-to-date whenever necessary. Every entry in the book shall be signed and dated by the surveyor, and countersigned and dated by the manager.

Preparation of plans by surveyors - (i) Every plan and section, and tracing thereof, prepared under the regulations shall be prepared by or under the personal supervision of the surveyor.

(2) Every plan or section, or any part thereof, prepared by or under the supervision of a surveyor shall carry thereon a certificate by him to the effect that the plan or section or part thereof is correct; and shall be signed and dated by the surveyor and countersigned and dated by the manager on every occasion that the plan or section is brought up-to-date.

(3) Every tracing of a plan or section or of any part thereof shall bear a reference to the original plan or section from which it was copied and shall be certified thereon by the surveyor to be a true copy of the original plan or section. The certificate shall be signed and dated by him.

(4) if the surveyor fails or omits to show any part of the workings or allows the plans or sections to be inaccurate, he shall be guilty of a breach of the regulations. Nothing shall, however, exempt the owner, agent, or manager of their responsibility to ensure that every plan or section prepared, kept or submitted under the regulations or by any order made thereunder is correct and maintained up-to-date as required thereunder.

Plans to be checked on change of ownership or on re-opening, etc. - (1) When there is a change in the ownership of a mine, or where a mine or part thereof is re-opened, or where in any mine or part thereof it is intended to start any extraction or reduction of pillars, the owner, agent and manager shall ensure that the plans or sections of the mine or part thereof are accurate. If any doubt arises as to the accuracy of the plans and sections in any respect he shall have accurate plans and sections prepared afresh before any drivage or other work of developement or of extraction or reduction of pillars is commenced.

(2) If the Regional Inspector is of the opinion that any plan or section prepared, kept or submitted under the regulations is inaccurate, he may, by an order in writing, require a fresh

survey made and a new plan or section prepared within such time as he may specify therein.

(3) If the plan or section required to be prepared is not prepared within the time specified in the order or to the satisfaction of the Regional Inspector, or the plan or section is not prepared or brought up-to-date as required under the regulaitons, he may get the plan or section prepared by any other agency; and the cost thereof, as certified by the Chief Inspector, shall be defrayed by the owner of the mine and recoverable from him as an arrear of land revenue.

VIRIETY OF PLANS AND SECTIONS

A great variety of plans are required to be maintained at a large modern mine under the various Regulations and Rules, and these include -

1. Surface plan : Showing every surface feature such as telephone, telegraph or power transmission line, watermain, tramline, railway, road or river. watercourse, reservoir, tank, borehole, shaft and incline openings, opencast workings, subsidence, building on surface, benchmark together with its R. L. above Mean Sea Level, contours at vertical intevals not exceeding 5 metres.

On the plan the grid lines should be marked in squares of 250 metres sides.

2. Underground Plan : Showing (i) the position of workings belowground, every borehole and shaft with depth, incline opening, cross-measure drift, goaf, fire-stoppings or seal, water dam with dimentions and particulars, pumping station and haulage roadways; (ii) every important surface features within the boudaries, such as railway, road, river, stream, watercourse, tank, resorvoir, opencast workings, building which is within 200 metres of any part of the workings measured on the horizontal plane; (iii) the general direction and rate of dip of the strata; (iv) Section of the seam; (v) position of every roll, washout, dyke and every fault with the amount and direction of its throw; (vi) surface contours of vertical intervals not exceeding 5 metres; (vii) spot levels on the floor of the workings along all haulage roadway junctions and in case of tramming levels every 150m apart, at the end of headings which have been discontinued temporarily or permanently; (viii) a permanent bench mark on the surface together with its height above Mean Sea Level; (ix) the workings and other features, both above and belowground, of all adjacent mines situated within 60 metres, measured on any plane, of the boundary claimed by the owner of the mines; (x) warning lines indicating the outlines of bodies of water and unconsolidated strata; and (xi) an abstract of all statutory restrictions in respect of any specified workings.

3. Vertical Mine Sections : Showing a vertical projection of the mine workings, where a seam has an average dip of more than 30° from the horizontal.

4. Ventilation Plan and Section : Showing the system of ventilation in a mine and in particular the general direction of air current, every point where quantity of air is measured, every air-crossing, ventilation door, stopping and every principal device for regulation and distribution of air, every fire-stopping and its serial number; every room used for storing inflammable material; the position for fire-fighting equipment; every water dam with dimensions and other particular or construction; every pumping, telephone and first-aid station; every haulage and travelling roadway. The position of stone dust barriers (with type and date

of construction) should also be indicated on the plan.

5. Combined Section or Sections : In case of coal seams within 9 metre of each other showing the workings of different seams in different colours.

6. Conbined/Composite Plan : Showing the extent of workings of all the seams lying within the boundary of the mine, in different colours.

7. Geological Plan : Geological plan of the area of the leasehold, on a scale having a R. F. 5000 : 1, or any other suitable scale, showing outcrops of seam together with rate and direction of dip, concealed outcrops, faults, dykes, rolls, washouts, sills, burnt coal and other geological features in the area.

8. Rescue Plan/Tracing : At least 3 in number showing all details required on Ventilation plan.

9. Water-danger Plan : For mines where exists danger due to flooding, showing highest known flood level, and all low lying lands likely to be affected by high floods, warning lines to draw visual attention to dangers of inundation arising out of surface water, unconsolidated starta, water-bearing strata and underground water.

The grid lines shall be marked on the plan in squares of 250 metre sides.

10. Abandoned Mine Plan : Showing all details as required for underground plans.

11. Stone Dusting Plan : Showing underground workings, divided into zones for stone dusting purposes, and stone dust barriers, maintained on a scale having a R. F. of 2400 : 1.

12. Sampling (zoning) Plan : Showing underground workings and clearly indicating the zones for sampling purposes, each not longer than 150m, and sections in each zone not longer than 50m each. Every zone shall be marked with a distinct number and every section with the code letter a, b or c, in a systematic manner.

13. Fire-fighting Plan : Showing the location of the fire-fighting stations under-ground, the layout of the fire-fighing pipe line and tapping points and other features required to be shown on the ventilation plan. Sufficent number of copies of the fire-fighting plans should be kept available at suitable places both on suface and under-ground.

14. Joint Survey Plan : Showing workings and features prescribed for under-ground plans both above and belowground of all adjacent mines as are situated within 60 metres measured on any plane of the boundary claimed by the owners of the mine.

15. Electrical Plan : Showing position of electrical equipments together with their specifications, the position of cables together with their construction and capacities.

16. Subsidence Plan and Sections : Showing observation points on the Test lines, depillared area and/or area under extraction, geological distrurbances, extent of surfase subsidence, method of extraction, section of the strata overlying the seam under extraction indicating (i) the thickness and nature of overlying strata, (ii) the conditions of the overlying seams, (iii)

angle of draw, and (iv) amount of subsidence from time to time.

17. **Sketch Plan of Systematic Support Rules :** In respect of areas, where Systematic Support Rules are in force, giving details of the support rules.

18. **Manpower Distribution Plan :** The number of persons employed in every district, and other places belowground on any one day of the month shown on a sketh plan. The plan shall indicate (i) the month of which the plan relates (ii) a single line sketch of the main roadways, in different colours when more than one seam, (iii) the name of districts, with letter reference. (iv) the number of persons employed in each district category wise, and (v) the summary of the total number of persons employed underground.

19. **Accident Plan :** Accident site plan, when any accident occurs in a mine resulting in loss of life or serious bodily injury to any person, indicating the name of the district, place, date and time of accident, position of the deceased/injured persons, other relevent details reqruired to illustrate the accident, and the cause of accident in brief.

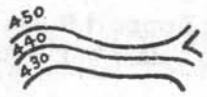
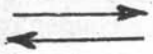
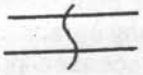
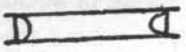
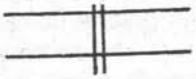
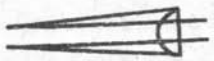
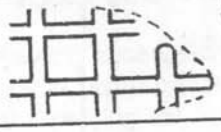
20. **Any other plan or Section as required by the Regional Inspector of Mines.**

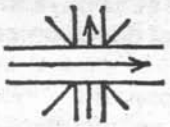
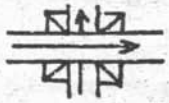
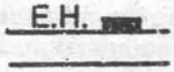
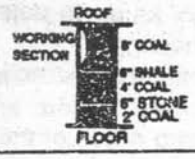
SECOND SCHEDULE

(Coal Mines Regulations, 1957)

CONVENTIONS FOR PREPARING PLANS AND SECTIONS

Name	Symbol	Remarks
FAULT	FAULT 30.3M ↓	in Red, showing the amount & direction of throw
DYKE OR OTHER INTRUSION	DYKE	In Green
GOAF		
SUBSIDENCE	SUBSIDENCE	In Red
BENCH MARK	B. M.	

Name	Symbol	Remarks
SURFACE CONTOUR		In Burnt Sienna
UNDERGROUND SPOT LEVEL	+ 104.94	In Blue
WATER DAM		In Red
DIRECTION OF AIR CURRENT		Intake in Blue Return Red
BRATTICE		In Red
DOORS		In Red
BRICK/STONE OR CONCRETE VENTILATION STOPPING		In Red
BOUNDARY OF LEASE HOLD		In Red
UNDERGROUND COAL BARRIER		In Green
SHAFT		
ABANDONED SHAFT		
INCLINE		
ABANDONED INCLINE		
PILLARS AND GALLERIES		
DRIFT		In Burnt Sienna showing gradient in Black.
QUARTERLY SURVEY LINE		

Name	Symbol	Remarks
STAPLE SHAFT		In Red should state distance up and down to all insets
ABANDONED STAPLE SHAFT		In Red
FIRE DAM, SEAL OR STOPPING		In Red
EXPLOSION PROOF STOPPING		In Red
AIR CROSSING		
EXPLOSION PROOF AIR CROSSING		
REGULATOR		In Red
AUXILIARY FAN		In Red
TELEPHONE		In Red
UNDERGROUND FIRST-AID STATION		Thick cross in Red
ENGINE HOUSE OR ROOM		
BORE HOLE		Should show serial number & diameter
SURVEY LINES & STATIONS		In red
SECTION OF SEAM		

STATUTORY PPROVISIONS

1. When a mine is abandoned : Where any mine is abandoned, two true copies of the up-to-date plan and section of the workings of the mine maintained under clauses (b) and (c) of Reg 59 (1) of the Coal Mines Regulation, 1957 shall be prepared by the surveyor for submission to the Director General of Mines Safety within 30 days after the abandonment. Every such copy shall show the bearing and distance of at least one of the shafts or openings of the mine from a trijunction or revenue pillar or from any other prominent and permanent surface feature, the position of all water dams built belowground (with dimensions and other particulars of construction) and also the spot levels at the ends of workings.

2. When ownership of the mine changed : The surveyor shall ensure that the plans and sections of the mine or part are accurate. If any doubt arises as to the accuracy of the plans and sections in any respect, he shall have accurate plans and sections prepared afresh before any drivage or other work of development or reduction of pillars is commenced.

3. When an old mine is re-opened : Plans and sections of the mine is checked to ensure their accuracy. In case of any doubt as to their accuracy, accurate plans and sections shall be prepared afresh.

4. When a tracing of Plan is prepared : Every tracing of a plan or section or any part thereof shall be prepared by or under the personal supervision of the surveyor, and shall bear a reference to the original plan or section from which it was copied and shall be certified thereon by the surveyor to be a true copy of the original plan and section. The ceritificate shall be signed and dated by him.

5. When workings are approaching the mine boundary : The surveyor shall record, in a bound-paged book kept for the purpose, the full facts when workings of the mine have approached to about 75 metres from the mine boundary. Every entry in the book shall be signed and dated by the surveyor and countersigned by the manager.

6. When doubts arise concerning the accuracy of plans : Any doubts which may exist concerining the accuracy of the plans or sections prepared under the regulations shall be recorded by the surveyor in the bound-paged book kept under regulation. The entry shall be signed and dated by him and countersigned and dated by the manager of the mine.

7. When it is proposed to start extraction of pillars : The surveyor shall ensure that the plans or sections of the mine or part are correct. If any doubt arises as to the accuracy of the plans and sections in any respect he shall have accurate plans and sections prepared afresh before any work of extraction or reduction of pillar is commenced.

The application, for permission in writing of the Chief Inspector of Mines, shall be accompanied by two copies of an up-to-date plan of the area where the pillars are proposed to be extracted or reduced showing -

(i) the proposed extent of extraction or reduction of pillars, and the manner of extraction;

(ii) the thickness and depth of the seam;

(iii) the rate and direction of dip;
(iv) the nature of roof; and
(v) the line of extraction passing over full pillars.

8. When working under railway, roads, etc. :

(i) No workings shall be made and no work of extraction or reduction of pillars shall be conducted at, or extended to, any point within 45 metres of any railway, or any public work, road, building or other permanent structure not belonging to the owner of the mine, without the prior permission in writing of the Chief Inspector and subject to such conditions as he may specify therein.

(ii) Every application for permission shall be accompanied by two copies of plan showing -

(a) the position of workings in relation to the railways, roads, etc.
(b) the manner in which it is proposed to carry out the intended operations, and
(c) the limits to which it is proposed to carry out the said operations.

A copy of the application and the plans shall also be sent to the railway administrations concerned in case of railways and such other authorities as the Central Government may direct by order in other cases.

An offset plan on a scale having a R. F. of 1:500 should also be maintained in the mine.

9. When working below river, canal, etc. : No working shall be made in any mine vertically below any part of river, canal, lake, tank or other surface reservoir, or any spot lying within 15 metres from either bank of the river or canal, or from the boundary of the lake, tank or other surface reservoir except with the prior permission of the Chief Inspector and subject to such conditions as he may specify therein.

The application for such permission shall be accompanied by 2 copies of a plan and section showing :

(i) existing position of the workings,
(ii) proposed layout of the workings,
(iii) depth of the workings from the surface,

Where sand, alluvium, etc. are lying in the course of a river, etc., the depth from the surface shall be reckoned from the surface of the hard ground underlying such sand or alluvium. To ascertain the hard bed-rock a borehole section has to be included in the application.

(iv) all faults dykes, and ohter geological disturbances,
(v) position and depth of any goaves in the neighbourhood, and
(vi) such other particulars as may affect the safety of the mine or of the persons employed therein.

All the above workings shall also be clearly demarcated underground. An offset plan on a scale having R. F. 1:500 should be maintained in the mine.

10. When workings are approaching disused or water-logged workings : The surveyor shall record in a bound-paged book kept for the purpose the full facts when workings of the mine have approached to about 75 metres from the disused or water-logged workings. The entry shall be signed and dated by the surveyor and countersigned and dated by the manager.

Every application to the Chief Inspector for the extension of any working which has approached within a distance of 60 metres (the shortest distance measured in any direction -

horizontal, vertical or inclined) of any disused or abandoned workings (not being workings which have been examined and free from accumulation of water), whether in the same mine or in the adjoining mine, or where any heavey seepage of water, which is not normal to the seam is noticed, shall be accompanied by 2 copies of a plan and section showing :

(i) the outlines of all such abandoned or disused workings, which are approaching the said workings;
(ii) the depth of such disused or abandoned workings;
(iii) the outlines, the layout and the method of the proposed working for which permission is sought; and
(iv) the faults, dykes, etc. in relation to the working.

11. When it is intended to construct a water dam in any mine : The owner, agent or manager shall give in writing not less than 14 day's notice of such intention to the Regional Inspector. Every such notice shall be accompanied by two copies of plans and sections showing the design and other details of the proposed construction (including the depth to which it is recessed into the roof, floor and sides, and materials to be used in its construction).

12. When working is more than one section in a seam : Every application for prior permission from the Chief Inspector of Mines for multi-section and contiguous workings, i.e, workings lying within 9m of each other shall be accompanied by two copies of a plan showing

(i) the proposed layout of the workings;
(ii) a section of the seam/seams;
(iii) depth of the seam/seams from the surface;
(iv) the rate and direction of dip;
(v) the proposed dimension of pillars and galleries in the seam or section; and
(vi) the thickness of the parting between the seams or sections.

An offset plan showing the size of pillars and galleries in both the sections, should accompany the application to facilitate ascertaining the coincident verticality of pillars.

CHAPTER - III

LINEAR MEASUREMENTS

★★★

METHODS OF LINEAR MEASUREMENTS

There are various methods of making linear measurement and their relative merit depends upon the degree of precision required. These methods may be divided under three main heads -

1. **Direct Measurements :** Distances are actually measured on the ground with the help of a chain or a tape or any other instrument.

2. **Optical Methods :** Observations are taken through a telescope and distances are calculated, such as tacheometry or triangulation.

3. **Electronic Methods :** Distances are mesured with instruments that rely on propagation, reflection and subsequent reception of either radio or light waves. The various instruments used under these methods are Geodimeter, Tellurometer, Decca Nevigator, Lambda Position Fixing System. Geodimeter is based on the propagation of modulated light waves and the other three instruments use radio waves for distance measurements.

DIRECT MEASUREMENTS

Several methods are available for measuring distances directly -

1. **Pacing :** The method consists in counting the number of paces between two points of a line. The length of the line is then computed by knowing the average length of the pace.

2. **Passometer :** It is a watch-like instrument operated by motion of the body to record automatically the number of paces, thus avoiding the monotony and strain of counting the number of paces. It is carried in pocket or attached to one leg of the person measuring the distance travelled by him.

3. **Pedometer :** The pedometer is a device similar to the passometer. It may be adjusted to the length of the pace of the person carrying it to register the total distance covered by any number of paces.

4. **Odometer and Speedometer :** The odometer is an instrument for recording the number

of revolutions of a wheel. It is fitted to a wheel which is rolled along the line whose length is required. Knowing the circumference of the wheel, the distance may be calculated. If the route is smooth, the speedometer can be used to measure the distance approximately.

5. Perambulator : It resembles a single bicycle wheel to register on the dials automatically the distance traversed by it.

6. Time Measurement : Distances are roughly determined by time intervals if the average time per kilometre for the person at walk is known.

7. Chaining : Chaining is a term which is used to denote measuring distance either with a chain or tape. For work of ordinary precision, a chain can be used, but for higer prescision a steel tape is invariably used. The distance determined by chaining form the basis of all surveying.

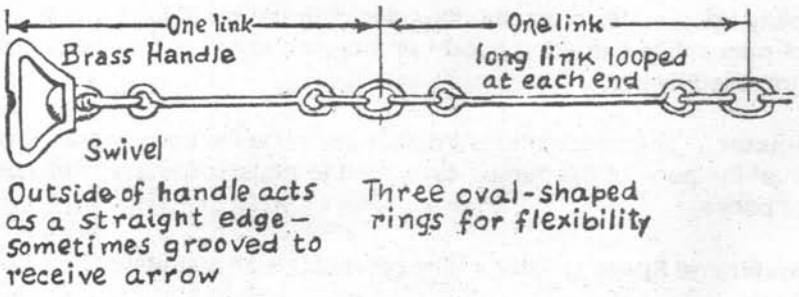
INSTRUMENTS FOR CHAINING

The instruments used for the determination of the length of a line by chaining are -

1. Chain
2. Arrows
3. Tapes
4. Ranging Rods
5. Offset Rods
6. Cross-staff or Optical Square
7. Plumb bob
8. Pegs
9. Wooden mallet

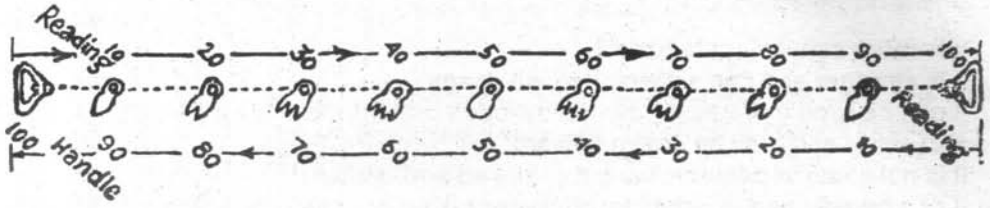
1. CHAIN :

The chain consists of a fixed number of straight links of galvanised mild steel wire 4mm (or 8 S.W.G) in diameter. The ends of each link are bent into a loop and connected together by means of three oval or circular rings. These rings provide flexibility of chain and make it less


One link
One link
Brass Handle
long link looped at each end
Swivel
Outside of handle acts as a straight edge – sometimes grooved to receive arrow
Three oval-shaped rings for flexibility

liable to become kinked. The joints of the rings are usually open but they can be welded so as to avoid the chain from wear and tear due to stretching. The ends of the chain are provided

with brass handle at each end with swivel joint, so that the chain can be turned without twisting. The length of a link is the distance between centres of two consecutive middle rings, while the length of the chain is measured from the outside of one handle to the outside of the other handle.



Sequence of tellers (tallies)

Types of Chain : With the introduction of Metric System in India, survey chains are divided into two types -

1. Metric Chains
2. Non-metric Chains.

METRIC CHAINS :

The chains in which the unit of measurements is taken in metres is called metric chains. These chains are generlally available in lengths of 5, 10, 20 and 30 metres. A 20m chain is divided into 100 equal parts whereas a 30m chain is divided into 150 equal parts. Each part is known as link and the distance between the centres of two consecutive rings is equal to 20 cm. To enable to measure the fraction of a chain easily brass tags (also called tallies, tablets, tellers) are provided at every 5m length and small brass rings at every metre length except where tallies are attached. The tallies are of different shapes i.e. the first tally from either end is simply a single prong; the second, again from each handle has two prongs; and the centre one, is simply a circular, or oval, disc. The tallies are marked with letters 'm' in order to distinguish them from non-metric chains. The full length of the chain is marked on both the handles.

STEEL BAND OR BAND CHAIN :

The steel band or band chain consists of a long narrow strip of a blue steel having uniform width of 12 to 16 mm and thickness 0.3 to 0.6 mm. It is available in length of 20 m or 30 m or may be made to any specified length. Brass studs are fixed at every 20 cm length and numbered at every metre. The first and the last 20 cm length graduated in cm and mm. Alternately, instead of putting brass studs, a steel band may be etched in metres, decimetres and centimetres on one side and 0.2 m links on the other side. For convenience in handling and carrying, steel bands are wound on specal steel crosses or metal reels having a large diameter of at least 50 cm. These steel band or bands made from invar are used for precise surveys e.g, base line measurement in triangulation survey, as they are not much affected during working but they should be used with care.

The advantages of steel band are that it is light in weight and practically unalterable in length, except in so far as it is affected by temperature changes. The degree of accuracy obtainable therefore very high. **The disadvantages** are that it is somewhat awkward to han-

dle and liable to kink and break. It must also be cleaned and oiled when out of use, unless made of stainless steel.

Comparision between Ordinary Chain and Steel Band : The practical advantages of the chain as compared with the steel band are -

1. It is more convenient to handle.
2. It is stronger and can withstand rough usage.
3. It can be read more easily on muddy roads and from either end. It is thus reversible and either end may be drawn forward.
4. It is not liable to become twisted or kinked and broken.
5. It can be adjusted in length by adding or taking out one or more connecting links. It can be repaired in the field.

The disadvantages are :

1. It is less accurate in its measurements.
2. It is heavy and cumbersome to carry and takes more time to spread.
3. It sags considerably when suspended.
4. Due to large number of wearing surfaces it is too much liable to increase in length by stretching when in continued use. It also gets shortened due to bending of the links and mud adhering to the rings. Frequent testing for standard length is therefore necessary.
5. It does not pass easily when drawn through hedges, stones and sleepers.
6. It is useless for precise work where allowance for temperature changes must be made.

In general, it might be said that a chain is only suitable for routine surveying and that a steel band is essential for important work.

NON-METRIC CHAINS

The chains in which other than metric units are taken are called non-metric chains. Since the introduction of metric units in our country non-metric chains are no longer in use. In F. P. S. System the following chains are in use -

(a) Engineer's Chain : The length of this chain is 100 ft and is divided into 100 links. At every 10 links (10ft) brass tallies are provided. The tallies for 10 and 90 have one prong, for 20 and 80 have 2 prongs, for 30 and 70 have 3 prongs, for 40 and 60 have 4 prongs, and the central tally for 50 links is round to represent 50 ft length.

(b) Gunter's Chain : A Gunter's chain or Surveyor's chain is 66 ft long and consists of 100 links, each link being 0.66 ft or 7.92 inches long. The length of 66 ft was adopted to facilitate land measurement since 10 square chains are equal to 1 acre. Again, 10 Gunter's chain = 1 furlong and 80 Gunter's chain = 1 mile.

(c) Revenue Chain : It is 33 ft long chain with 16 links, each link being $2\ ^1/_{16}$ ft long. The chain is mainly used for measuring field in cadastral survey.

2. ARROWS

The marking pins which are made of stout steel wire 4 mm (or 8 SWG) in diameter and having length equal to 40 cm in all are called arrows. One end of the arrow is pointed and the other end is bent into a circular ring for making it easier to carry. Arrows are used to mark the end of chain lengths during the process of chaining. When several lengths are required to measure a line, the total number of arrows inserted equals the number of full chains.

3. TAPES

For taking fracional distances, less than a chain length, tapes are used. It is also used for taking offset distances from a chain line. Tapes are classified according to the material of which they are made such as follows -

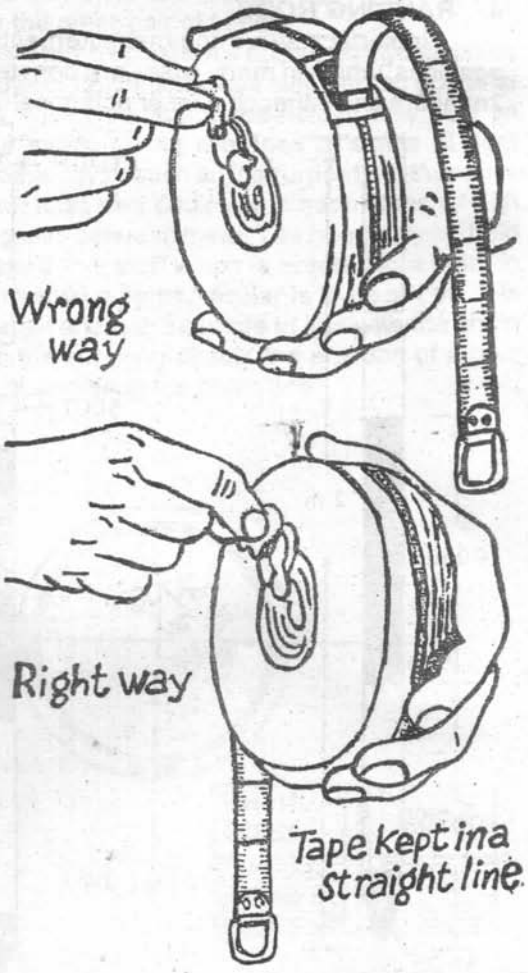
(a) Cloth or linen tape.
(b) Metallic tape.
(c) Steel tape.
(d) Invar tape.

Cloth or Linen tape : It consists of a varnished strip of closely woven linen 12 to 15 mm wide. The end of the tape is provided with a brass ring whose length is incluced in the tape length. It is available in lengths on 10 m, 20 m and 30 m. Each metre length is divided into 100 parts to read centimetres. It is available in a leather case with a winding device. Linen tapes are used for measuring offsets. Very accurate measurements with these tapes are not possible due to the following reasons -

(a) It cannot withstand much wear and tear as its length gets altered by stretching.
(b) It is easily affected by dampness and thus shrinks.
(c) It is likely to twist.
(d) It is not so strong.
(e) It shortens by sagging in windy weather.

Metallic Tape : It consists of a varnish strip to linen or cloth 12 to 15 mm wide having very fine brass or copper or bronze wires woven into it to prevent it from elongation and twisting. Metallic tapes are light and flexible and are not easily broken. They are particularly suitbale in cross-sectioning and in some methods of topography where small errors in length of the tape are of no consequence.

Steel Tape : A steel tape consists of a light steel strip of width 6 to 10 mm. It is more accu-



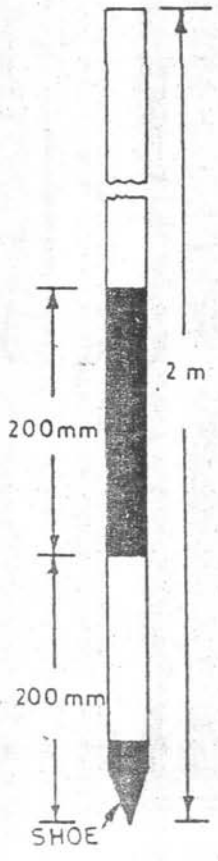
Steel tape being wound in

rately graduated. Each metre length is divided into 200 equal parts to read upto 5 mm. The first 10 cm length is graduated upto millimetres. A steel tape is lighter and delicate, so it cannot withstand rough usage. It is easily kinked and thus breaks. Steel tapes are generally superior to a cloth or linen tape or a metallic tape for most of the linear measurements that are made in surveying.

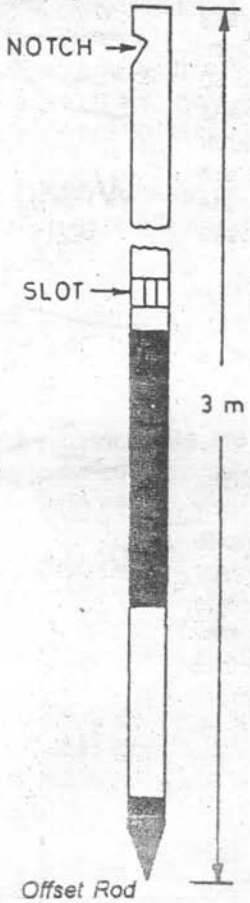
Invar Tape : Invar tapes are used mainly for linear measurements of a very high degree of precision. It consists of a strip which is made of an alloy of steel (64%) and nickel (36%). It possesses a very low co-efficient of thermal expansion with an average value of 0.0000005 per 1°F (0.9×10^{-6} per 1°C). Bands and wires made of invar enable base lines to be measured very much more rapidly and conveniently. Invar tape are more expensive, much softer and are more easily deformed than steel tapes. Invar tapes are normally 6 mm wide and are available in lengths of 20, 30 and 100 m.

4. RANGING RODS

In surface surveying measurements have to be taken along lines to definite and fixed positions, which in many cases are denoted by 3 cm diameter wooden rods having a length of 2m or 3m and either circular or octagonal in section. For penetration into the gruound, one end is fitted with a pointed metal shoe, and to make each rod conspicuous to the observer, it is alternately painted black, white and red in lengths of 20 cm. When used at long distances they are provided with flags or jhandies at the top - 30 to 50 cm squares and of two colours, generally red and white or yellow and white. These rods may also be made of hollow steel pipes and sometimes of bamboos.



Ranging Rod



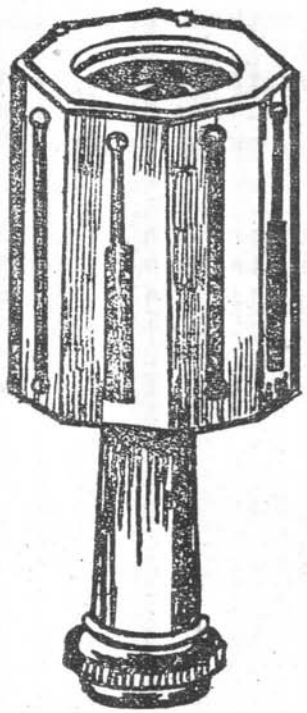
Offset Rod

In case of ranging very long lines, **Ranging Poles** are used which are similar to ranging rods except that they are longer (4 to 8 m) and of greater diameter (6 to 10 cm). On some poles vertical slots at eye level (cut in pairs at right angles to each other) enable an observer to set out two lines approximately at right angles to each other. Metal supports may be used for holding the ranging pole when the ground is too hard for insertion; heavey wooden blocks, with a hole pierced through the centre to receive the pole, answer equally well.

5. OFFESET RODS

An offset rod is simillar to a ranging rod. At the top of it a hook or a notch is provided to help in pulling or pushing

the chain through a hedge or other obstruction. It may be used for measuring offsets roughly.



Octagonal Cross-staff for angle 45° and 90°

6. CROSS-STAFF & OPTICAL SQUARE

The surveyor is often called upon to make use of a right angle. In the absence of a miner's dial or a theodolite, a right angle may be set out with fair accuracy (a) by the use of a cross-staff, or an optical square or (b) by means of the chain alone.

CROSS-STAFF : The simplest form of **Open Cross-staff** consists of four sight vanes so arranged that the two side lines cross each other at right angles. The instrument is fixed on a pointed rod or light tripod set up at the point in the line from which a right-angle is to be set out. One pair of vanes is aligned along the line and the right-angle is then ranged out with the other pair of vanes.

A French Cross-staff Consists of a hollow octagonal brass tube. It has an alternate vertical sight slits and an opposite vertical window with a vertical horse-hair or fine wire on each of the four sides. giving two lines of sights at right angles to each other. With such an instrument it is therefore possible to set off lines perpendicular to each other, or with angles of 45 degrees between them. The head may be held in hand or fastened to a staff which is erected to a point in the chain line at which a perpendicular is to be set off. It is then turned till a slit and hair sight are in line with the chain line, after which the surveyor directs the erection of a pole in the line at right angles to the chain line.

In the more modern form of this instrument the brass cylinder divided horizontally into two parts and both provided with slits for sihgting purposes. The lower portion is graduated in degrees and the upper portion may be rotated relatively to the lower portion by means of a thumb screw provided below, the angle passed is indicated by a vernier on the upper part. In this way, any angle may be set out, although not with a high degree of accuracy.

OPTICAL SQUARE : This is a simple, compact and small pocket instrument that can be held in the hand when observations with it are to be made. It is capable of being tested

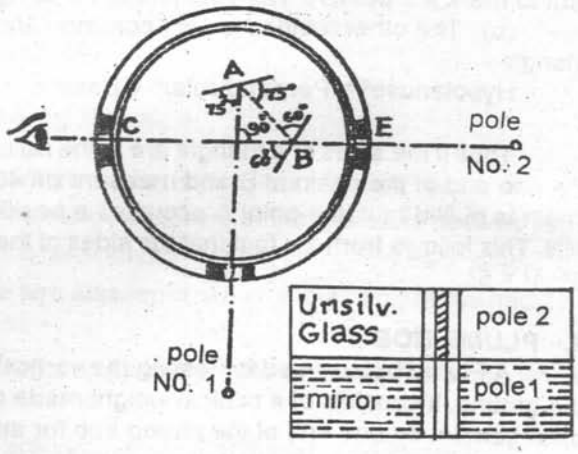


Diagram of the Optical Square

and adjusted. The principle of this handy instrument depends on the fact that a ray of light, on striking a plane mirror not at right angles to the ray and reflected into another plane mirror, undergoes deviation and the angle of deviation equals twice the angles between the two reflecting surfaces.

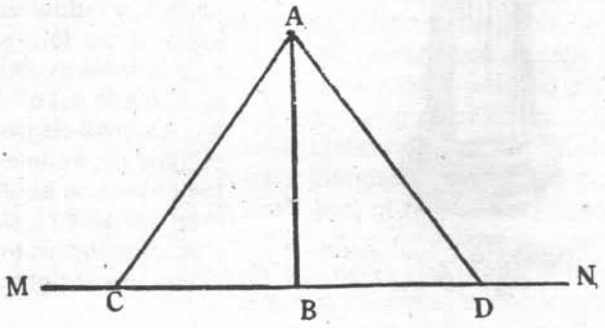
The instrument consists of a small circular box, about 65 cms in diameter and 1.3 cm deep, containing two mirrors which are fixed so that angle between the planes of the surfaces is 45°. The mirror at A is completely silvered, and the mirror at B is silvered over the lower half, the upper part being plain glass.

To construct a right angle, the instrument is held vertically above the point in the chain line at which it is proposed to erect a perpendicular. The instrument is turned until, by looking through the sighting hole at C, the observer gets the image of pole No.1 in chain line on lower part of mirror B. On looking through the plain unsilvered portion of mirror B, and through the vision hole E, pole No. 2 is fixed in the required direction by directing the assistant to move the pole either right or left until it coincides with the reflected image of pole N0. 1. Then the offset is perpendicular to the chain line.

SETTING OUT RIGHT ANGLES WITH A CHAIN OR TAPE

The methods to set out a right angle by using only a chain or tape are -

(a) Let a line BA is to be set out from the point B and perpendicular to the chain-line MN. At first points C and D are taken at equal distances from the point B.



Now the two ends of the tape are held at C and D, and holding the middle point the tape is pulled taut to mark the point A. Then the line BA is at right angles to MN.

(b) The other method is very common and it depents on the fact that in a right-angled triangle -

$$\text{Hypotenuse}^2 = \text{Perpendicular}^2 + \text{Base}^2$$

Thus if the sides of a triangle are in the ratio 3:4:5 then it must be a right angled triangle. Fix one end of the chain at B and measure off 40 links to C. Then fix the 80 link point of the chain is pulled taut, the point A occupies a position such that the line BA is perpendicular to MN. This follows from the fact that the sides of the triangle BAC are in the ratio 30: 40: 50, and $3^2+4^2 = 5^2$

7. PLUMB BOB

A Plumb bob is used for testing the verticality of an object and for transfering points on the ground. It consists of a conical weight made of any metal preferably of brass. There is an arrangement on one end of the plumb bob for attaching a thread.

While chaining along sloping ground, a plumb bob is required to transfer the points of the ground. It may also be used to make ranging poles vertical and as centering aid in theodo-

lite, plane table and a variety of surveying instruments.

8. PEGS

Pegs are used for marking the positions of survey stations and other stations required for survey purposes. These are made of hard wood preferably of Sishum or Babul. They are usually square in cross-section having side from 3 cm to 5 cm and length 15 cm. They are tapered at one end. Pegs are driven into the ground with the help of a wooden mallet (hammer) and kept about 4 cm projecting from the surface.

Sometimes they may be round in section. For soft ground pegs 40 to 60 cm long are used. Iron pegs about 1 cm to 2 cm diameter may be used in harder ground.

9. WOODEN MALLET

It is used for driving the peg into the ground. Wooden mallet or hammer is made of hard wood - 7 to 10 cm diameter at ends, about 20 to 25 cm in length and is provided with a wooden handle about 45 cm long.

UNFOLDING AND FOLDING OF A CHAIN

The surveyor should learn the technique of the unfolding and folding of the chain :

Unfolding : To lay out the chain on the ground -

(1) Remove the leather strap. Take both the handles in the left hand, and hold the remainder of the chain in the right hand.

(2) Holding the handles in the left hand, throw the chain forward on the ground with the right hand.

(3) Lay the handles on the ground so that the two halves of the chain lie side by side.

(4) Leaving one handle on the ground, pick up the other handle and walk forward until the chain is fully extended.

(5) See if there are any kinks or bent links.

This operation is also called as *Undoing the chain.*

Folding : After the work is over, the chain should be folded into a bundle. To do this -

(1) Bring the two handles together on the ground so that the chain lies doubled up. Pull the chain from the middle point to make the two halves straight alongside each other.

(2) With the right hand, pick up the two adjacent links on either side of the middle and place them obliquely in the left hand.

(3) Likewise, take the next pair of links and continue the procedure until the whole of the chain has been rolled up.

(4) Fasten the leather strap firmly around the waist of the links so formed.

This operation is known as *doing the chain.*

USE OF THE CHAIN

In measuring lines, the direction of which is marked by poles set up at the ends of the lines, at least two persons, a leader and a follower, are required. The leader, in possession of 10 arrows, pulls out the chain in the direction of the far station, and, having been ranged in line with the pole at the far station, inserts an arrow in the ground to mark the end of the chain. The leader then proceeds further along the line until another chain has been marked out, when the follower will pull up the arrow which was left to mark the end of the first chain.

When the leader arrives at the far station he holds the handle of the chain at the foot of the station pole, and the follower reads the distance from the last inserted arrow to the station to which distance he adds the number of arrows in his possession. Thus, if in measuring a line the follower collects 8 arrows and the distance from the last inserted arrow to the station pole is 26 links, the length of the line will be 826 links. A station peg may be inserted in the ground at the end of every 10 chains to prevent confusion and to form a useful check.

In measuring a line great care should be taken to secure proper alignment. Intermediate poles should be used in ranging long lines. The chain should be pulled taut and the arrows should be inserted perpendicularly into the ground.

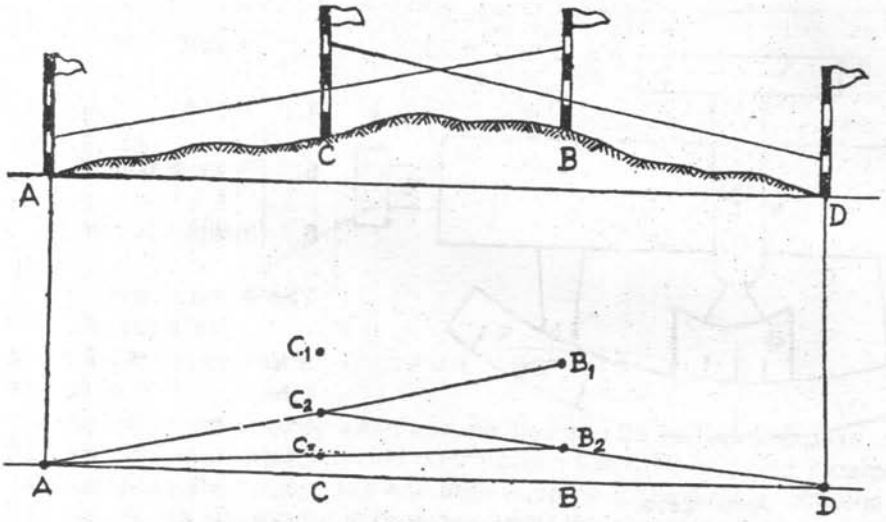
RANGING A LINE

This means, in its simplest sense, placing poles, or other suitable markers, at the terminals of the line. Underground, the end of the lines are marked with lamps on tripod when a survey is in progress, but are converted to permanent marks with pins or nails let into wooden plugs in the roof.

Ranging intermediate poles : When surface lines are of considerable length, it becomes necessary to increase the number of poles so that the measurements may be made in a straight line. To align fresh poles, the surveyor may use a line ranger, or he may line them in by eye. To do this he stands close behind the pole at rear end and looking towards the pole at forward end directs, with signals, an assistant to set up an intermediate pole, so that it obscures that at forward end. It is necessary to set up more than one intermediate pole, they are all set up so that the latest one inserted obscures the previous one inserted.

Double Ranging : In ranging over hilly ground, it often happens that owing to the configuration of the surface, the pole at the far end of a line is invisible from the rear station. In such cases the normal method of direct ranging is inapplicable, and an indirect method known as Reciprocal Ranging or Double Ranging is used.

Two polemen, B and C are stationed on the intervening high ground so that both their poles are visible from D and A but not necessarily in line with them. Let their initial unaligned positions be B_1 and C_1 as shown in the figure. B_1 now directs C_1 into the line $B_1 C_2 A$. And C_2 directs B_1 into line $C_2 B_2 D$. This is repeated until all four poles are in a straight line as shown in figure plan.



Double Ranging

MEASURING ON SLOPING GROUND

Plans have to be plotted from horizontal measurements, and consequently all slope distances must be reduced to their horizontal equivalent before they can be poltted. There are two methods of doing this : (a) Stepping and (b) Measuring on the slope.

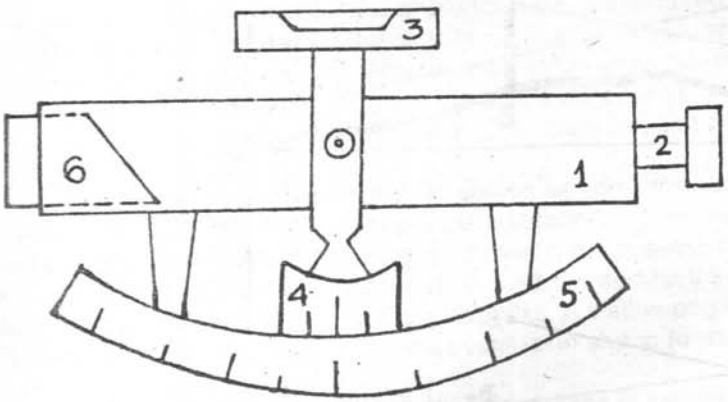
(a) (i) Stepping Downhill : In this case, the routine of the follower continues to hold his end of the chain or tape on the ground. The leader elevates his end to the same level as the follower's This is difficult to do, and an observer should decide when the tape is horizontal. The leader is aligned and the elevated position is transferred to the ground by means of a plumb-bob, or drop-arrow, the later being an ordinary arrow with a few centimetres above the pointed end.

(ii) Stepping uphill : This is the reverse of the above process, but is more difficult because the follower's end is now off the ground, and he must plumb the end over the mark and align the leader simultaneously.

Whether "stepping" uphill or downhill, it must be remembered that the steeper the slope the shorter is the extended length of tape, and 3m may sometimes be the maximum length to avoid the effects of sag caused by the weight of the chain or tape. The elevation should rarely exceed 2m per step.

(b) MEASURING ALONG THE SLOPE

Besides the slope length, the angle of inclination must also be measured. This may be done by using a small hand instrument such as clinometer or Abney Level.



Abney Level

1. Sighting Tube
2. Eye-piece
3. Bubble Tube
4. Vernier
5. Semi-circular Arc
6. Inclined Mirror

The Abney Level

The body of the instrument consists of a sighting tube, about 12 cm long, square in section, and carryng at one end a draw-tube with a pin-hole sight or eye-piece, and at the other end a horizontal wire and a highly polished mirror which slopes 45° to the horizontal. Against the side of the sighting tube is fixed a graduated semi-circular arc. A spindle running through the arc centre carries a bracket on the side holding a spirit level, and on the other an index vernier which measures the amount of inclination.

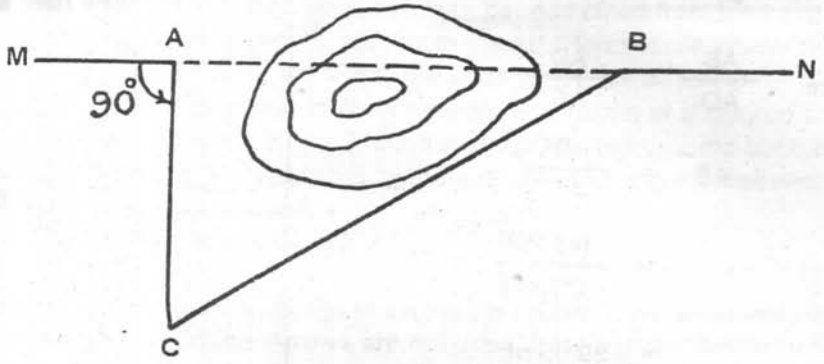
The arc is graduated in degrees with zero at the bottom and 60° at the sides. When the sighting tube is tilted to observe an inclination, the angular amount of slope is measured by operating a slow-motion screw which brings the bubble of the spirit level to the centre of its run, and simultaneously moves the vernier along the graduations. The observer when using the tube is able to see the position of the uncovered side of the bubble in reflector set beneath it in the tube. A horizontal line is scribed accross the transfer axis of the mirror, and the line must bisect the bubble and, at the same time, coincide with the horizontal cross-wire fixed in the end of the tube when an observation is being made. To determine the angle of inclination, the surveyor, standing on the line of slope, directs an assistant to hold a pole some distance away on the same slope. The Abney Level is placed to the eye so that the bubble is apparent in the mirror, and he tilts the tube to observe the pole; the amount of tilt as degrees of arc is measured by the vernier on the graduated scale. The cosine of this angle when multiplied by the slope length will give the horizontal plan length.

OBSTACLES IN CHAIN LINE

It may be found impossible to measure some lines directly between the stations because of obstacles being in the way. These may be buildings, lakes, rivers or pit rubbish heaps. In general, the cases that may arise may be divided into three classes :

1. An obstacle that can be seen across as well as chained around e.g. a pond.

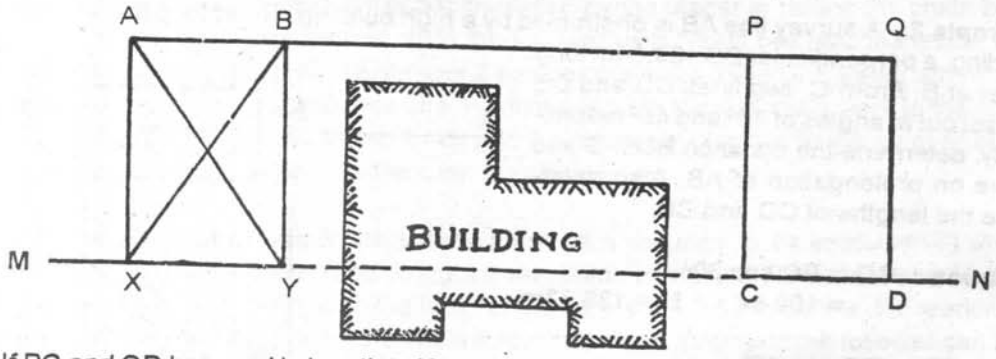
Two convenient points A and B are selected on the chain line MN on opposite sides of the pond as shown in the figure. Perpendicular AC of such a length is set that CB clears the obstacle.



AC and Cb are measured, Then AB = $\sqrt{BC^2 - AC^2}$

2. An obstacle that can not be seen across, but can be chained around, e.g. a building.

Two points X and Y are selected on the chain line MN. At X and Y perpendiculars XA and YB of equal lengths are erected and their equality is checked by diagonals XB and YA, which must be equal. The line AB is prolonged past the obstacle and two points P and Q are selected on it. Perpendiculars PC and QD are set out at P and Q respectively.



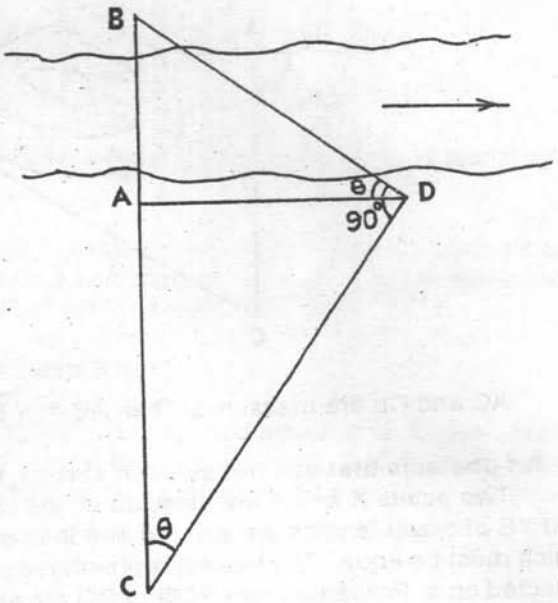
If PC and QD be equal in length to XA, the points C and D must be on the chain line MN. The length of BP will give the length of YC.

3. An obstacle that can be seen across but not chained around, e.g. rivers, narrow valleys, or ravines.

In such cases, the use of similar triangles solves the problems. All the methods assume that the ground in the vicinity is reasonably flat. If the ground is rugged, an angular instrument must be used to get an accurate result.

Example 1. A and B are two points on the opposite banks of a river along a chain line CAB which crosses the river at right angles. The surveyor selects a point D which is 48.20m from A along the bank and sets a perpendicular CD on the line BD. If the distance CA is 59.50 m, determine the distance AB.

Solution : The figure showing the points is roughly plotted. The triangles ACD and ABD are similar.



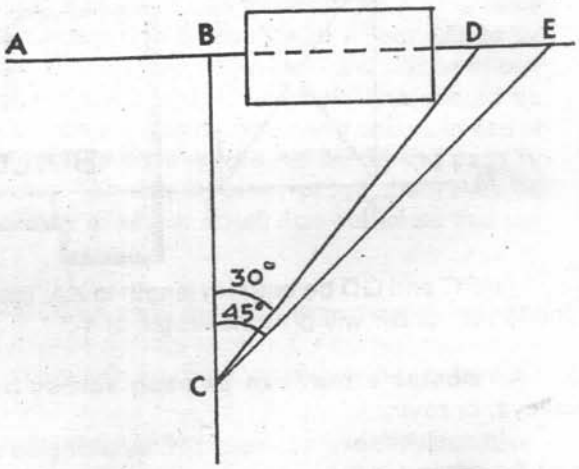
$$\text{Hence,} \quad \frac{AB}{AD} = \frac{AD}{AC}$$

$$\text{or} \quad AB = \frac{AD^2}{AC}$$

$$= \frac{(48.20)^2}{(59.50)}$$

$$= 39.05 \text{ m.}$$

Example 2. A survey line AB is obstructed by a high building. To prolong the line beyond the building, a perpendicular BC 109.84m long is set at B. From C, two lines CD and CE are set out at angles of 30° and 45° respectively. determine the distance BD if D and E are on prolongation of AB. Also determine the lengths of CD and CE.



Solution : CD = BC sec 30°
= 109.84 x 1.5 = 126.32m

CE = BC sec 45°
= 109.84 x 1.41 = 154.87m

BD = BC tan 30°
= 109.84 x 0.577 = 45.47m

CLINOMETER

The angle of slope, or the gradient, of the surface of the gruond may be determined by means of a semi-circular protractor having a plummet suspended from the centre, the whole being mounted on a suitable stand which may consist of a rectangular board. The edge of the

stand which is parallel to the straight limb of the protractor, and adjacent to the curved limb, is placed on the ground. The angle of slope can then be read direct from the point of contact of the plumb line with the protractor or scale. Another useful form of clinometer is that which is embodied in some forms of rules. On one of the arms of the rule a spirit level is fixed, and the joint is graduated to read degrees, to determine the inclination of a line, on the surface or underground, the rule is placed on the ground and opened out until the bubble of the spirit level is at the mid of its run. The angle can then be read from the graduated circle of the joint.

MEASURING LINES UNDERGROUND

In measuring lines in the workings of a mine the procedure is necessarily a modification of that employed at the surface. Arrows are not used beloweground because the floor is so hard that it is quite impossible to insert them in the ground. The leader, therefore, marks the end of the chain with a piece of chalk. It will be convenient to mark the end of the chain on the rails, if these are dry, but if wet, a flat stone may be requisitioned upon which to mark the end of the chain.

Setting out from a dial station the leader pulls the chain until he is stopped by the follower who calls on him to "mark one". The leader then marks the end of the chain and also the number of chains measured, on the rails or on a stone. If a stone is used the surveyor must send some one to see that it is not displaced by the leader in pulling the chain beyond that point. At the end of the second chain the follower calls "mark two" and the leader marks the position of its end and puts the number 2 besides the mark. When the leader arrives at the forward station the follower will measure the distance from the last chain mark to that station, and to determine the total distance he will add the fractional distance to the number of chain indicated on the rails or stone, as the case may be.

In carrying out an important survey great care requires to be exercised in order to secure that lines will be measured straight from station to station. The leader should always be put into correct alignment with the lamp at the far station, and to do this, the leader holds his lamp directly over the end of the chain and in such a manner that the follower can see it, without having his view of the station obstructed. To mark the end of the chain a hard stone should be selected, or a plumb line should be suspended from the roof so that point of plummet is directly above the mark of the leader's lamp. If the mark is made on a stone two lines should be drawn, one in the chain line and the other at right angles, so that point of intersection marks exactly the end of the chain.

TESTING AND ADJUSTMENT OF CHAIN

The length of every chain may change due to constant use and variation in temperature, etc.

The chain may decrease in length due to (i) links becoming bent, or (ii) dirts getting into

joints.

The chain may increase in length due to the following reasons :

(i) Stretching of the links and joints,
(ii) Opening of the ring-joints,
(iii) Wear during rough handling in pulling it through hedges and fences and over rough ground.

As per Indian Standards Specitications (I.S.) every metre length shall be accurate to within ± 2 mm when measured with a pull of 8 kg and checked by steel tape or steel band standardised at 20° C. The over all length of chain shall be within the following limits :

For 20 m chain, error shall not be more than $\pm$ 5 mm.
For 30 m chain, error shall not be more than $\pm$ 8 mm.

It is therefore necessary to check the length of the chain and adjust to its proper length before commencing the measurements and also at frequent intervals, Before testing the chain, the bent up links should be straightened and the chain should be made from mud-sticking.

TESTING THE CHAIN

Folowing methods may be used for testing the chain :

1. Comparing with Chain Standard or with a Test Gauge.

The chain standard is established on a long floor, such as verandh floor of a large building or railway platform, by means of a standard steel tape or invar tape. A **test gauge** for the purpose may be formed upon an even surface, preferably a pavement, a distance of one chain length should be measured accurately with a steel tape. The whole line should be divided into links, and marks should be cut in the pavement with hammer and chisel. In lieu of a pavement a standard chain length may be ranged out on level ground by driving pegs to mark the ends of the chain and the 10-link divisions. The sub-divisions of the chain may be marked by driving nails into the pegs, or better still by driving in small iron plates so that the edges just lie on the surface of the pegs.

A permanent test gauge should be established at every mine surveyor's office so that the chain may be tested frequently. In carrying out large surface survey it is sometimes desirable to make a test gauge close to the scene of operations, so that before commencement of each days work the chain may be applied for testing.

2. Comparing with a steel tape or invar tape which is kept in the office only for this purpose.
3. Comparing the chian with a precisely graduated levelling staff.

ADJUSTING THE CHAIN

When on testing it is found that the length of chain is not correct, it may be adjusted as under :

1. *When the chain is too long -*
   (a) By closing up the joints of the rings opened.

(b) By hammering back to the original shape of the flattened or oval rings.
(c) By replacing some of the worn rings .
(d) By the adjustable links at the handles.
(e) By replacing some of the larger rings by smaller ones.

2. *When the chain is too short* -
(a) By straightening the bent up rings.
(b) By flattening some of the rings.
(c) By replacing some of the smaller rings by larger ones.
(d) By the adjustable links at the handles.
(e) By adding new rings as required.

The adjustment of the chain should be carried out in such a way so that the position of tags may not change.

ERRORS IN CHAINING

The difference between a measured quantity and the true value of the quantity measured is the **ture error** of the measurement. Since the true value of the quantity is never known so the true error is diffcult to be known. However, the duty of the surveyor is to do measurements so precisely that these should be correct within certain limits of error.

SOURCES OF ERRORS

In chaining of distances there are three chief sources of errors -

1. Instrumental Errors : The errors which occur due to faulty adjustments or inaccuracy in making the instruments are callled instrumental errors. For example a chain or tape being too long or short.

2. Natural Errors : The errors which arise due to natural effects such as due to heavy winds, refraction, variation in temperature, etc. are called natural errors.

3. Personal Errors : The errors which arise due to carelessness of the surveyor such as applying unequal tension in the chain, non-alignment of a chain line, not removing parallax or sighting the object incorrectly, etc. are called personal errors.

CLASSIFICATION OF ERRORS

The chief sources of errors may also be classified under the following heads -
(a) Cumulative Errors.
(b) Compensating Errors.
(c) Mistakes.

CUMULATIVE ERRORS

These are liable to occur in the same direction and tend to accumulate or add up as the chaining proceeds. These errors may be regarded as **Positive if** the measured length is in

excess of the true length, or **negative if** the measured length is less than its true length. Cumulative error is proportional to the total length measured which seriously affects the accuracy of the work. They may occur due to -

(i) Incorrect length of the chain or tape.

The error will occur if the chain or tape used is either too long or too short which may be due to opening of rings or joints, kinking, missing of rings, temperature variation, etc.

Prevention : The error can be minimised by correcting the chain before commencment of work. The ture distance may also be worked out from formula -

$$\text{True distance} = \frac{\text{Incorrect length of chain}}{\text{Correct length of chain}} \times \text{measured distance}$$

(ii) Sag on the Chain or tape :

When measurements are made on sloping gruoud by stepping method and no sag correction is applied.

Prevention : The error can be minimised by decreasing the length of steps and pulling the chain firmly or by supporting the chain at suitable intervals.

(iii) Non-alignment of the chain line :

In such case the measured length along the chain line will be more and the error will always be positive.

Prevention : Proper alignment of any chain line is essential to prevent the error.

(iv) Chain or tape not being straight :

This may be due to high winds or bushes interveing the chain or tape may bend them resulting measured distance to be more than actual.

Prevention : All obstacles like bushes, etc on the way should be cleared and proper pull should be applied.

(v) Variation in temperature :

The length of chain or tape may be increased or decreased due to rise or fall in temperature.

Prevention : The errors may be minimised if the work is done in the morning upto 10 O'clock and then in the evening after 5 P.M. for precise measurements, the temperature may be recorded and correction may be applied.

COMPENSATING ERRORS : These are liable to occur in either direction and have tendency to compensate the result. Compensating error is proportional to the square root of the length measured. These are not as serious as cumulative errors. Compensating errors may occur due to the following causes -

(i) Incorrect holding of the chain : The follower may not hold the end of the handle

exactly over the mark left by the leader thus placing it sometimes a little ahead or in rear of the mark.

(ii) **Incorrect placing of an arrow :** The leader may fix an arrow sometimes inside the handle and sometimes outside the handle.

(iii) **Incorrect plumbing :** In stepping method of measurements when some crude method is followed than using a plumb bob.

(iv) **Unequal pull applied to chain :** The length of a chain may increase when the pull applied is more than the standard and it will decrease when the pull is less.

DISPLACEMENT OF ARROWS : Mistakes are errors which are committed due to inexperience, carelessness and poor judgement of the surveyor. If not detected they may seriously affect the final result. These can be avoided by rechecking the records in the field itself as the work proceeds. The following are the mistakes which are commonly made by the surveyor during chaining -

(i) **Displacement of arrows :** Sometimes the arrow is displaced from its position, it may then become difficult to replace it correctly. The end of each chain length should therefore be marked both by arrow and by a cross (x) mark on the ground.

(ii) **Mistake in counting the Chain Length :** The follower may forget to pick up the arrow fixed by the leader, thereby measuring the length of line short by one chain. This can be checked by counting the number of arrows both in the hands of the follower as well as the leader and full number should be equal to ten.

(iii) **Reading from the wrong end of the chain :** Sometimes the measurement is read from the wrong end of the chain. It can be avoided by noticing the position of central tag and then the final reading noted.

(iv) **Reading numbers wrongly :** Reading the tape upside down for example 6m for 9m or vice versa.

(v) **Wrong booking :** Incorrect booking e.g. 28.46 for 28.64. The chainman should call out reading loudly and the surveyor should repeat it before booking.

(vi) **Miscellaneous :** Such as calling numbers wrongly e.g. calling 60.3 as "sixty three" or reading wrong foot marks e.g. 25.47 for 24.47.

CORRECTION FOR ERROR DUE TO INCORRECT CHAIN OR SCALE

If the length of the chain used in measuring length of the line is not equal to the true length or the designated length, the measured length of the line will not be correct and suitable correction will have to be applied. If the length of the chain used is more than its true

length, the distance measured will be less and the error in such case will be negative and correction will be positive. Similarly, if the chain is too short, the measured distance will be more, the error will be positive and the correction will be negative.

CORRECTION FOR ERRONEOUS LENGTH OF CHAIN

Let L = True or correct length of the chain or tape
L' = Incorrect (or actual) length of chain or tape used.

1. Correction to measured length :

$$\text{Correct length of line} = \text{Measured length of line} \times \frac{L'}{L}$$

2. Correction of area :

$$\text{True area} = \text{Measured area} \times \left(\frac{L'}{L}\right)^2$$

3. Correction of volume :

$$\text{Correct volume} = \text{Incorrect volume} \times \left(\frac{L'}{L}\right)^3$$

CORRECTION DUE TO USE OF WRONG SCALE

Sometimes the length of a line is measured from the plan or map already drawn by using a scale. If the scale used is wrong, the length obtained is incorrect. The true length of the line is then given by the relationship :

$$\text{True length} = \frac{\text{R.F. of wrong scale}}{\text{R.F. of correct scale}} \times \text{measured length}$$

Where R.F. means representative fraction of the scale.

CORRECTION DUE TO SHRUNK SCALE

Sometimes due to variation in atmospehric conditions the already drawn plan shrinks. It becomes necessary to find the shrunk scale of the plan.

Let the original scale as written on the plan is equal to 1 cm = x m.

Measure the distance between any two points on the shrunk plan by using this scale. Compare the length of the line with the actual distance between the two points. Then :

Shrinkage ratio or shrinkage factor

$$= \frac{\text{Shrinkage length (by measurement)}}{\text{Actual length}}$$

$\therefore$ **Shrunk scale = Shrinkage factor x Original scale.**

For example if the shrinkage factor is $^{10}/_{11}$ and the original scale is 1 cm = 10 m (or its R.F. = $^{1}/_{1000}$), the shrunk scale will have a R.F. = $^{10}/_{11} \times {}^{1}/_{1000} = {}^{1}/_{11000}$ or 1 cm = 11 m.

Example 1. The length of a line measured by a 20 m chain was found to be 350 m. Calculate the true length of the line if :

(a) The chain used was 10 cm too long.
(b) The chain used was 4 cm too short.

Solution : Incorrect measured distance = 350 m.

(a) Incorrect length of Chain (L') = 20+0.10 = 20.10 m.
Correct length of Chain (L) = 20 m.

Hence, Correct length of the line = incorrect distance x $\frac{L'}{L}$

$$= 350 \times \frac{20.10}{20} = \mathbf{351.75\ m}$$

(b) Incorrect length of Chain (L') = 20-0.04 = 19.96 m

Correct length of the line = Incorrect distance x $\frac{L'}{L}$

$$= 350 \times \frac{19.96}{20} = \mathbf{349.3\ m}$$

Example 2. The length of a line measured with 20 m chain was found to be 372 metres. The true length of the line was known to be 371 metres. Find the error in the chain.

Solution : Incorrect length of the line = 372 m
Correct length of the line = 371 m
Correct length of the chain = L = 20 m
Incorrect length of the chain = L'

$$\therefore \text{ Correct length of the Chain} = \frac{\text{Correct length of line} \times L}{\text{Incorrect length of line}}$$

$$= \frac{371 \times 20}{372} = 19.95\ \text{m}$$

Error in the chain = 20 - 19.95 = 0.05 m
= **5 cm too short.**

Example 3. A 20 m chain was found to be 10 cm too long after chaining a distance of 1500m. It was found to be 18 cm too long at the end of day's work after chaining a total distance of 3000m. Find the true distance if the chain was correct before the commencement of the work.

Solution : For first 1500 metres -

$$\therefore \text{ Average error } = e = \frac{0 + 10}{2} = 5 \text{ cm} = 0.05 \text{ m}$$

$$L' = 20 + 0.05 = 20.05 \text{ m}$$

$$\text{Hence correct distance for 1500 m} = \frac{20.05}{20} \times 1500 = 1503.75 \text{ m}$$

For next 1500 metres -

$$\text{Average error } = e = \frac{10 + 18}{2} = 14 \text{ cm} = 0.14 \text{ m}$$

$$\therefore L' = 20 + 0.14 = 20.14 \text{ m}$$

$\therefore$ Hence correct distance for second length of 1500 m

$$= \frac{20.14}{20} \times 1500 = 1510.5 \text{ m.}$$

Total distance = 1503.75 + 1510.50 = **3014.25 metres.**

Example 4. A 30 m chain, which was correct before the commencement of work, was found to be 10 cm too long at the end of day's work after measuring a distance of 3845m. Find the correct length of the line.

Solution : An error of 10 cm is gradual; therefore

$$\text{Mean incorrect length of the chain} = L' = \frac{30 + 30.1}{2} = 30.05\text{m}$$

Correct length of the chain = L = 30m
Incorrect measured distance = 3845m

$$\therefore \text{ Correct distance} = \text{Incorrect distance} \times \frac{L'}{L}$$

$$= 3845 \times \frac{30.05}{30} = \mathbf{3851.41m}$$

Example 5. The distance measured between two points by a 20m chain was 1345m and when measured by a 30m chain was 1350m. If the 30m chain used was 1 link too short what correction is required in the 20m chain ?

Solution : The distance measured by 30m chain = 1350 m

The Correct length of the chain = L = 30m = 150 links.
The Incorrect length of the chain = L' = 150 - 1 = 149 links

Hence correct length of the line = Incorrect distance x $\frac{L'}{L}$

$= 1350 \times \frac{149}{150}$ **= 1341m**

In the first case the distance measured by a 20m chain = Measured incorrect distance = 1345 m

$\therefore$ Incorrect length of chain = $\frac{\text{Correct distance x L}}{\text{Incorrect distance}} = \frac{1341 \times 20}{1345} = 19.94\text{m}$

Hence the error in 20m chain = 20 - 19.94 = 0.06 m = **6 cm too short .**

Example 6. The distance was measured between two stations on a plan drawn to a scale of 10 m to 1 cm and the result was 1210.8 m. Later it was discovered that the distance was measured by using a scale of 12 m = 1 cm by mistake. Find the true distance between the stations.

Solution : Wrong scale used 12m = 1 cm, its

Representative fraction (R.F.) = $\frac{1}{1200}$

Correct scale is 10 m = 1 cm $\therefore$ R.F. = $\frac{1}{1000}$

Hence correct distance = $\frac{\text{Incorrect R.F}}{\text{Correct R.F}}$ x measured length

$= \frac{1}{1200} \times \frac{1000}{1} \times 1210.8 =$ **1009 m**

Example 7. The plan of an old mine survey plotted to a scale of 50 m to 1 cm was found to have shrunk so that a line originally 20 cm long was 19.4 cm. There was also a note on the plan that the 20 m chain used was 0.1 m too long. If the area of the plan measured now is 150.44 square cm, find the true area of the survey on the ground.

Solution : Measured area on the plan = 150.44 sq. cm. Since the plan has shrunk an original area of (20x20) sq. cm now measured (19.4x19.4) sq. cm.

Original area on the plan $\left(\frac{20}{19.4}\right)^2 \times 150.44 = 159.89$ sq. cm.

The scale of the plan being 50 m to 1 cm, the area on the ground = 159.89×50^2

= 399725 sq. m. The chain being 0.1 m too long, the actual length of the chain = **20.1 m.**

$$\text{True area on the ground} = \left(\frac{20.1}{20}\right)^2 \times 399725 = 403732.24 \text{ sq. m}$$

= 40.373224 hectares.

Example 8. The calculated length of a chain of a mine leasehold was 600 acres. On testing the chain was subsequently found to be 0.9 links too long. What is the correct area in acres ?
(Survey Comp. 1962)

Solution : Calculated area of leasehold = 600 acres.
Length of chain = 100 links
Actual length of chian = 100 + 0.9 = 100.9 links

$$\text{Hence, actual area of leasehold} = 600 \times \left(\frac{100.9}{100}\right)^2 = \textbf{610.85 acres}$$

Example 9. A survey was made with a chain 0.4 link too short and plotted to a scale of 1" = 400'.

The plan was found to have shruk so that a line originally 6" long was found to be 5.0". If the area of the plan as measured is 20 square inches, find the true area of the survey.
(Survey Comp. 1964)

Solution : Length of chian = 100 link
Actual length of chain = 100 - 0.4 = 99.6 link
Original length of a line = 6 in. Shrunk length = 5.8 in.
Present area measured on plan = 20 sq. in.

$$\text{Original area before shrinkage} = 20 \times \left(\frac{6}{5.8}\right)^2 \text{ sq. in.}$$

Plan of scale : 1 in = 400 ft

$$\text{Area of the plot} = 20 \times \left(\frac{6}{5.8}\right)^2 \times 400 \times 400 \text{ sq. ft.}$$

= 3424495 sq. ft.
But since the length of chain was erroneous

$$\text{Actual area of the plot} = 3424495 \times \left(\frac{99.6}{100}\right)^2 \text{ sq. ft.}$$

$$= \frac{3397154}{14400} \text{ Bighas} = 235.91 \text{ Bighas}$$

True area of survey is **235.91 Bighas.**

Example 10. The area surveyed by a 20m chain was plotted to a scale 1 cm = 50m and found to be 140 cm². It was found later that the chain used was 10 cm too long Find the true area of land surveyed in hectares.

Solution : Incorrect length of chain = L' = 20.1m
Correct length of chain = L = 20m
Incorrect area surveyed = 140 x 50 x 50 = 350000 sq. m

$$\text{True area} = \text{Incorrect area} \times \left(\frac{L'}{L}\right)^2$$

$$= 350000 \times \left(\frac{20.1}{L}\right)^2 = 353509 \text{ sq. m} = \textbf{35.3509 hectares}$$

Example 11. The area of the plan of an old survey plotted to a scale of 10 metres to 1 cm measures now as 90.5 sq. cm as found by a planimeter. The plan is found to have shrunk so that a line originally 10 cm long, now meaures 9.5 cm only. There was also a note on the plan that the 20m chain used was 9 cm too short. Find the true area of the survey.

(A.M.I.E. 1974)

Solution : Original length of a line = 10 cm
Shrunk length = 9.5 cm

Therefore, the line now 1 cm long was originally $\frac{10}{9.5}$ cm and the area of 1 sq. cm. at present was originally $\left(\frac{10}{9.5}\right)^2$ sq. cm.

Plan area = 90.5 sq. cm now, would therefore have been originally plotted

$$= 90.5 \times \left(\frac{10}{9.5}\right)^2 \text{ sq. cm.}$$

The scale of the plan is 1 cm = 10m and the area 1 sq. cm = 100 sq. m.
The measured area of the plot on the ground was

$$= 90.5 \times \left(\frac{10}{9.5}\right)^2 \times 100 \text{ sq. m} \qquad = 10027.701 \text{ sq. m}$$

The incorrect length of the chain = L' = 20 - 0.09 = 19.91m

$$\therefore \text{ True area of the survey} = \text{incorrect area} \times \left(\frac{L'}{L}\right)^2$$

$$= 10027.701 \times \left(\frac{19.91}{20}\right)^2$$

$$= 9937.65 \text{ sq. m} = \textbf{0.9938 hectares.}$$

HORIZONTAL EQUIVALENT OF A SLOPE LENGTH

If a distance is measured on a slope it can not be laid down directly on plan, for a plan shows horizontal distances only. In all such cases, the slope length must be reduced to the plan length for plotting purposes. There are various ways of calculating the horizontal equivalent of a slope length, depending on whether the angle of the inclination of the difference in level between two points on the slope is known.

If the angle of Inclination is known :

In this case, if A is the angle of inclination from the horizontal as measured with an angular instrument by the surveyor -

Horizontal length = Slope length x cos A

If the differnce in level is known :

Sometimes in the place of measuring the angle of inclination of the ground, the difference in the level between the points is measured with the help of a levelling instrument and the horizontal distance computed. Thus, if h is the difference in level then

$$\text{Horizontal length} = \sqrt{\text{Slope length}^2 - h^2}$$

HYPOTENUSAL ALLOWANCE :

If only the distance between the ends of a line is required, the horizontal distance may be computed after chaining is completed. If, however, a number of intermediate points have to be located, a correction may preferably be applied in the field at every chain length. The correction may be obtained as follows -

Let θ = the angle of a slope of the ground.
Horizontal length = 1 chain = 20m = 100 links.

Then the length of the line measured along the inclination (or inclined length)
= 100 Secθ links.
Hence the difference between the inclined length and its horizontal equivalent
= (100 secθ – 100) links or 100 (secθ – 1) links.

It therefore follows that the distance that must be measured on the slope in order that its horizontal equivalent should be equal to one chain or 100 links, is 100 secθ links. The arrow mark must therefore be put in advance of the end of the chain along slope by an amount 100 (Secθ – 1) links or metre. This excess amount 100 (secθ – 1) is known as the hypotenusal allowance. In the record book, the horizontal distance between the two points on slope can thus be directly recorded allowing for the slope correction.

Example 12. The distance between two points measured along a slope is 518 m. Find the horizontal distance between them if (a) the angle of slope between the points is 8°, (b) the difference in level is 78 m, (c) the slope is 1 in 4.

Solution : Let D = horizontal length
l = measured length

(a) $D = l \cos\theta = 518 \times \cos 8^0 = 512.96$ m

(b) $D = \sqrt{l^2 - h^2} = \sqrt{518^2 - 78^2} = 512.09$ m

(c) $D = \dfrac{518 \times 4}{\sqrt{17}} = \mathbf{502.63m}$

Example 13. Find the hypotenusal allowance per chain of 20m length if (a) the angle of slope is 10°, (b) the ground rises by 4m to 1 chain length.

Solution : (a) Hypotenusal allowance = 100 ($\sec\theta - 1$) links
= 100 (sec 10° – 1) links = 1.54 links = **0.31 m**
(b) $\tan\theta = {}^4/_{20} = {}^1/_5 = 0.2$

$\theta = 11^0\ 19'$

Hypotenusal allowance = 100 ($\sec\theta - 1$) links

= 100 (sec 11° 19' – 1) links = 1.983 links = **0.4m**

Example 14. If in a length of one 20m chain along a slope the ground rises 3.5m, find the angle of slope, and the hypotenusal allowance in cm per chain.

Solution : Let θ = the angle of slope.

Then $\sin\theta = \dfrac{3.5}{20} = 0.175$ $\theta = 10^0\ 4'\ 43''$

Hypotenusal allowance per 20 m chain
= 20 ($\sec\theta - 1$) = 20 (sec 10° 4' 43" – 1)
= 20 (1.0157 – 1) = 0.314m = **31.4 cm**

Example 15. A survey line was measured with a 20m chain on a falling gradient of 1 in 8 and found to be 12.48 chains. Later, however, it is found that the chain was 0.4 link too short. What length should be the line scale on a plan ?

Solution : The plan length is the true horizontal length of the line and this may be obtained from the measured length by applying the corrections for slope and error of the chain.

The gradient is 1 in 8, hence the horizontal length of the line

$= 12.48 \times \dfrac{8}{\sqrt{65}} = 12.3826$ chain

Again, the chain being 0.4 link too short, its actual length = 100 -- 0.4 = 99.6 links.
Hence the true horizontal length of the line.

$$= 12.3836 \times \frac{96.6}{100} = \textbf{12.34 chains.}$$

FINAL ERROR IN A LINEAR MEASUREMENT

The final error in a linear measurement is composed of two portions :
(a) Cumulative errors which are proportional to L and (b) compensating errors which are proportional to $\sqrt{L}$, Where L is the length of the line.

Example 16. A line 980m in length is measured with a 20m chain which is 0.02m too long, and error in marking a chain length is ± 0.03m. Calculate the amount of compensating and cumulative errors.

Solution : Compensating error of marking

$$= \pm 0.03 \times \sqrt{\frac{980}{20}} = \pm 0.21 \text{ m}$$

Cumulative error due to incorrect length

$$= 0.02 \times \frac{980}{20} = 0.98 \text{ m.}$$

Example 17. A line 270m long was measured with a 30m long tape which was 0.002m too long and the error in marking a tape length was ±0.002m. Calculate the amount of compensating and cumulative errors. **(Survey Comp . 1979)**

Solution : Compensating error of marking = $\pm 0.002 \times \sqrt{\frac{270}{30}}$

$$= \pm 0.002 \times \sqrt{9} = \pm 0.006\text{m}$$

Cumulative error due to incorrect length

$$= 0.002 \times \frac{270}{30} = 0.002 \times 9 = 0.018\text{m}$$

EXERCISES

1. The length of a line measured by 20m chain was found to be 350m. Find the true length of the line if (a) the chain used is 5cm too long, (b) if the chain used is 4cm too short, (c) if the chain is 0.4 link too long. **(Ans.** 350.87m, 349.3m, 351.4m)

2. The length of a line measured with a 20m chain was found to be 542.5 metres. The true length of the line was known to be 541 metres. Find the error in the chain. (**Ans.** 0.055m)

3. The distance measured along a slope of 12° was recorded to be 1648m. It was subsequently found that the chain was 10cm too short. Find the true horizontal distance.
(**Ans.** 1603.93m)

4. If in a length of one 20m chain along a slope the ground rises by 2m, find the angle of the slope and the hypotenusal allowance per chain. (**Ans.** 5°43', 9.98cm)

5. The distance measured along a slope between two points is 350 m. Find the horizontal distance between them when (a) the angle of slope is 15° (b) the difference in elevation is 10 m. (**Ans.** 338.07 m, 348.26 m)

6. The length of a chain line as obtained from the field book was 125m, but it was found that the chain used was incorrect. If the true length of the line was 115.94m, find the error in the chain if the chain used was 20m. (**Ans.** 15cm too long)

7. The distance between two stations was measured by a 20m chain and found to be 1000 metres. The same distance was measured with a 30m chain and found to be 996.17 metres. If the 20 m chain was 0.05m too short what was the error in 30m chain ?
(**Ans.** 0.03m too long)

8. The distance measured by a surveyor between two stations on a plan drawn to a scale of 20m = 1cm was 1080m. Later however it was discovered that he used a scale of 15m = 1cm. Find the true distance between the stations. (**Ans.** 1440m)

9. A 20m chain used for a survey was found to be 20.10m at the beginning and 20.30m at the end of the work. The area of the plan drawn to a scale 1cm = 10m was measured with the help of a planimeter and was found to be 38.42 sq. cm. Find the area of the field.
(**Ans.** 3919.22sq. m)

10. The area of the plan of an old survey plotted to scale of 1cm = 10m measures now as 108.8 sq. cm as found by a planimeter . The plan is found to have shrunk so that a line originally 10cm long now measures 9.6cm only. There was also a note that the 20m chain used was 8cm too short. Find the true area of the survey. (**Ans.** 10720sq. m)

11. A distance of 2000m was measured by a 30m chain. Later on it was detected that the chain was 0.1m too long. Another 500m (total 2500m) was measured and it was detected that the chain was 0.15m too long. If the length of the chain in the initial stage was quite correct determine the exact length that was measured. **(A.M.I.E., 1976)** (**Ans.** 2505.38m)

12. The area of the plan of an old map plotted to a scale of 10m to 1cm measures now 100.2sq. cm as measured by planimeter. The plan is found to have shrunk so that a line originally 10cm long now measures 9.7cm only. Further the 20m chain used is 8cm too short. Find the true area of the survey. **(A.M.I.E., 1973)** (**Ans.** 1.0564 hectares)

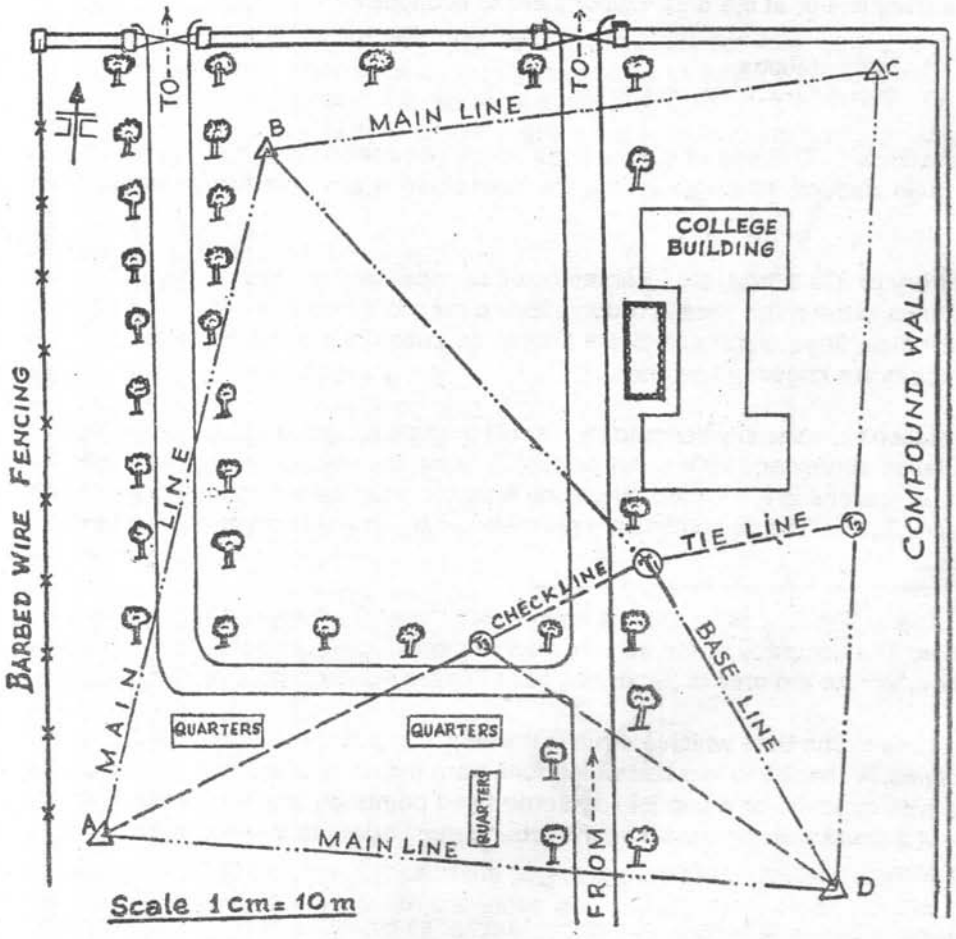
CHAPTER - IV

CHAIN SURVEYING

★★★

appropriate lengths from each end. The exact arrangement of triangles to be adopted depends upon the shape and configuration of the ground and natural obstacles met with. The three sides of a triangle being equally liable to error, and since the triangles are plotted by striking arcs and indefinite intersections will be obtained if any of the angles are very acute, it follows that the triangles should be as nearly equilateral as possible and such triangle being known as *well-conditioned or well-shaped triangle*. Ill-conditioned triangles having angles small than 30° or greater than 120° should always be avoided in chain-traingulation to minimise the errors in measurement and plotting.

In chain traingulation, the area to be surveyed is first divided into large triangles which are surveyed with greatest accuracy. Further the area is sub-divided into framework of trian-



Layout for a chain Survey

gles to get more details. In this way accumulation of error is prevented and there is better control. It is known as *Working from the whole to the Part*. On the other hand if the work is done from the part to the whole by dividing the whole area into smaller traingles and connect-

ing them together, the error will be magnified and uncontrollable at the end.

When planning the layout of the survey lines, aim at making the main framework as simple as possible and covering the whole area with as few triangles as possible. For a quardrilateral, the minimum number of lines to be measured is five but a sixth line should be measured as a check. Main line should be sited so as to run close to part of the detail to be surveyed and remaining features can be located by breaking down the framework with subsidiary detail lines beginning and ending at any measured points on the main network of triangles. Simple layout for a chain survey is indicated in the figure.

SURVEY STATIONS

An important point of the chain line is clalled *survey station*. It is either at the beginning of the survey line or at the end. Stations are of two types -

(i) Main stations.
(ii) Subsidiary or Tie staitons.

Main stations : The end of survey lines which command the boundaries of the survey are called *main stations*. The lines joining the main stations are called *main survey lines or chain lines*.

Subsidiary or Tie Stations : Sometimes it is necessary to select other points on the main survey lines to run more lines in order to locate the more interior details when they are distant from the main lines, such points are known as subsidiary or *tie stations*. The lines joining these points are known as *tie lines*.

Stations are usually denoted by a small triangle (△) round the station point in case of triangulation survey and by a small circle (O) round the station point in case of traverse survey. Main stations are denoted by capital letters or numbered alphabetically such as A, B, ...or 1, 2..... The tie stations are represented by small letters or otherwise as a, b,.....or T_1, T_2.....

Base Line : The longest chain line upon which the entire framework is made is known as a *base line*. The accuracy of the survey work depends upon precision in measurement of the base line. Where the area is large, two base lines in the form of letter X should be run.

Check Line : The lines which are run in the field to check the accuracy of the work are called *check lines*. A check line is a line which runs from the apex of a triangle to some fixed points on the side opposite or a line joining some fixed points on any two sides of a triangle. The length of a check line measured on the ground must agree to the length of this line as shown on the plan.

Preliminary Observations : In order to familiarise himself with the main features within the area to be surveyed, the surveyor should walk over the area and estimate the best positions for the survey stations and survey lines. This is known as *reconnaissance*.

SELECTION OF SURVEY STATIONS

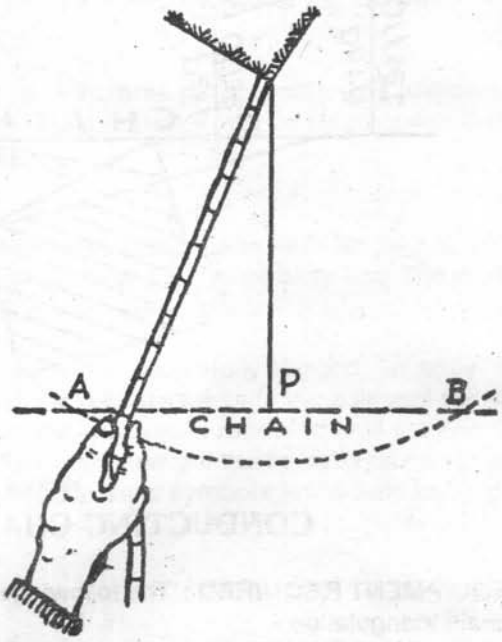
The following points should be considered while selecting survey stations :

1. Survey stations should be intervisible.
2. The line joining these station points should be clear straight lines for measurements and must run through as level ground as possible.
3. The main line should form well-conditioned triangle and each traingle should be provided with at least one check line.
4. The framework must have one or two base lines preferably selected on a level ground and should be divided into main triangles or traverses following the principle *working from whole to the part.*
5. Subsidiary or tie lines should be run if necessary to locate the interior details and to avoid long offsets.
6. Survey-lines should be so arranged as to avoid obstacles in ranging and chaining.
7. All the survey lines should be arranged to pass close to the surface features such as road, buildings, fences, etc. so as to avoid long offsets.
8. Station points should be so selected that the chain lines run at one side of to avoid inturruption of chain.

OFFSETS

The lateral distance of an object or ground feature measured from a chain line is called an offset.

Perpendicular Offsets : When the angle of offset from a point on the chain line is 90° it is called *parpendicular offset.* It involves less measuring on the ground. In most cases, offsets are perpedicular to survey line, and as right angles are usually estimated, then to preserve the accuracy of the plan, they should be as short as possible. Offsets exceeding 15m should be set out by means of a cross staff. A tape or an offset rod may be used for measuring offsets. Two measurments are necessary for offsetting, first the distance from the beginning of the chain line and secondly the distance from the chain line to the object.



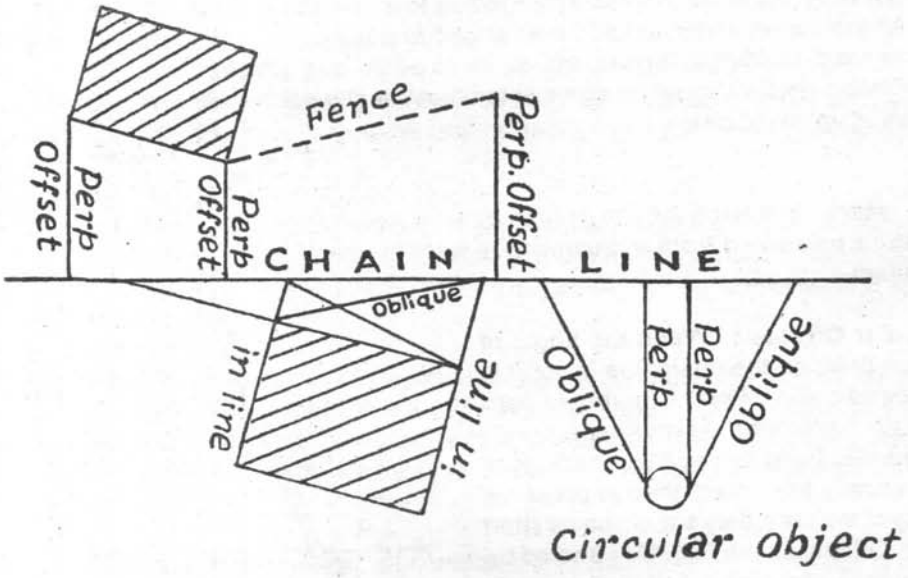
Offsetting from a point

The accuracy in measuring offsets is relative to the scale of plotting, the importance of the object required to be located and the length of the offset. As it is difficult to distinguish distances less than 0.25mm on paper, the limit of precision in plotting is 0.25mm. If the scale of plotting is 1cm = 10m, 0.25mm on paper will correspond to $\frac{0.25}{10} \times 10 = 0.25$m on the ground. Hence the offset should be measured to the nearest 25cm in this case.

OFFSETTING

The leader holds the tape end at the required position, the follower quickly swings the taut tape in an arc about the chain line and takes the mid-point between the arcs (shortest length of tape) as his perpendicular point.

Oblique Offsets : When an object has to be located with greater than the ordinary accuracy as is commensurate with the scale, or if it is a comparatively long distance away from the survey line, or should lie at an appreciable angle to the latter, then two non-perpendicular offsets are taken from different parts of the line, thus forming a triangle. These are *oblique offsets*, and are often used in conjunction with *in line offsets* which form continuation of fences, wall or sides of buildings. They not only fix position of a point but serve to indicate the directions of the feature between offsets.



Various Kinds of Offsets

CONDUCTING CHAIN SURVEY OF AN AREA

EQUIPMENT REQUIRED : The following is the list of equipment required for chain survey or chain triangulation -

(i) A Chain (20m or 30m) with 10 arrows.
(ii) A 10m or 20m metallic tape.
(iii) 6 to 8 ranging rods with flags and, 1 offset rod.
(iv) An Obtical Square or Cross-Staff.
(v) A plumb bob.
(vi) Pegs, wooden mallet, field glass, nails, chalk, knife, axe, stout string, etc.
(vii) Field-book, pencil, eraser.

FIELD WORK

A chain survey may be executed in the following steps -

(a) Reconnaissance of the area
(b) Marking and fixing survey stations
(c) Running survey lines and field notes
(d) Plotting or office work.

(a) Reconnaissance of the area

The priliminary inspection of the area to be surveyed is known as *reconnaissance*. By careful reconnaissance the shape and extent of the area to be surveyed can be obtained which will help to determine the difficulties in the work, time required for the survey and the intervisibility of the selected stations. A *key Plan* is prepared showing all the principal features on the ground, the position of station, chain lines, base line and tie lines. Intervisibility of stations is checked by ranging rods.

b) Marking Stations

The control stations, already proposed on the key plan are then marked with wooden pegs by driving them firmly into the ground and fixing nail on the pegs or by fixing ranging rods at each selected station. The survey stations are protected till the survey operation is completed. These stations may be permanent or temporary according to the nature of the survey.

All stations should be located by measurements from three permanent points and carefully recorded by means of a reference sketch. As the station marks may be displaced or lost, the reference sketches are very necessary.

c) Running Survey Lines and Field Notes

After completing the preliminary work, the direction of noth line with respect to any survey line is found and recorded. The chaining may be started from the base line. The work in running a survey line is two-fold : (i) to chain the line, and (ii) to locate the adjacent details.

The chain is strecthed along the line on the ground and carefully aligned. An arrow is inserted into the ground to mark the end of the chain. Offsets are taken to the adjacent object from the chain line. The chain is moved forward and the process is repeated until the end of the line is reached. The same process is followed for other lines. All the field measurements, sketches and other relevant notes are recorded using standard symbols in the field book for future reference and for preparing plans.

d) Plotting or Office Work

The surveyed lines, along with details are plotted on a drawing paper to a suitable scale in the office in accordance with their shape and size. North line and the position of stations with respect to reference objects are plotted first. The skeleton sketch of bigger triangles is then drawn showing the main survey lines. With respect to each survey line the plotting is carried out till all the survey lines are covered as recorded in the field book. The accuracy of the plotted work is checked from the rough sketch prepared, distances measured, check lines and tie lines. After checking and completing the plan, it may be inked. The title of the survey, the scale used, and the magnetic north with date must be recorded on the plan.

RECORDING FIELD NOTES

FIELD BOOK : The note book in which the chain or tape measurements, sketches and other field data are recorded for future reference is called *field book*. It is about 20cm x 12cm in size and opens lengthwise. There are two types of field books (i) Single line field book and (ii) Double line field book.

(i) Single line Field Book :

The type of field book in which a single red line is ruled down in the middle of each page is called *Single line field book*. This line represents the chain line, against which the length of line, chainages for offsets are taken. The space on either side of the line is utilised for drawing sketches along the chain line and distances of offsets are recorded opposite to the chainages to the left or to the right of the line as per actual position of the object in the field. It is used for a comparatively large scale and most detailed dimension work.

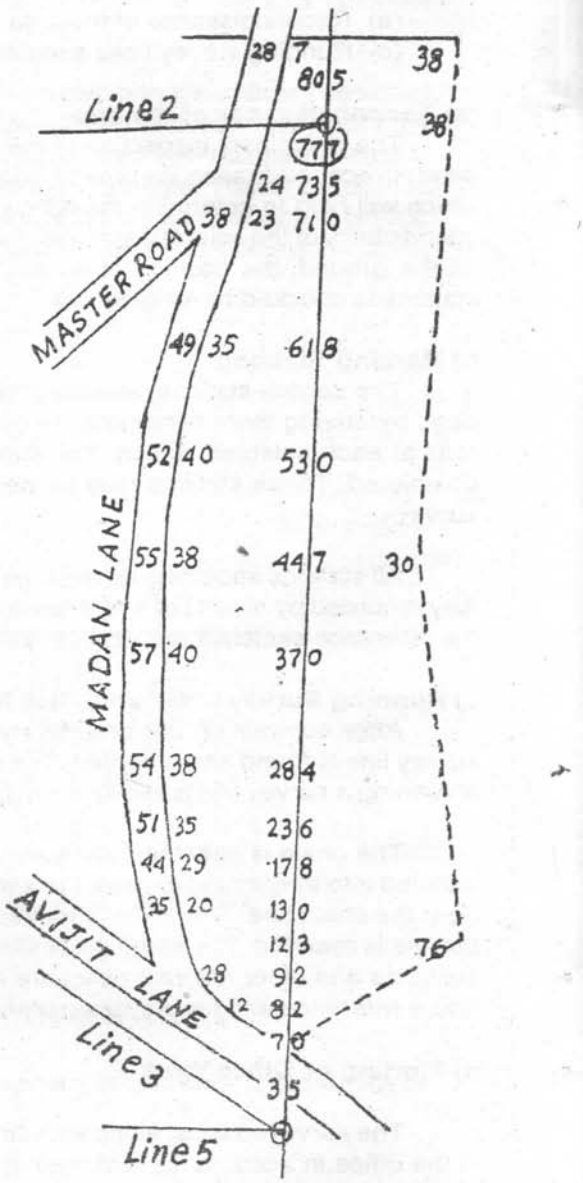
(ii) Double Line Field Book :

In this type of field book two red or blue lines spaced about 1.5 to 2cm apart are ruled down in the middle of each page. The double line field book is most commonly used for ordinary work, the distance along the chain line being entered between the two lines of the page.

Every chain line must be started from a fresh page in the field books.

Particulars of Chain Survey

At the beginning of a particular chain survey, the following details must be written in a field book :



Single Line Field Book

1. Date of Survey and name of surveyors.
2. General sketch of the layout of the survey lines and other features with a north line.
3. Page index of survey lines.

4. Location sketches of survey stations.

5. Name of the line.

6. Name of the station marked by a circle or triangle.

7. Bearing of the line, if measured.

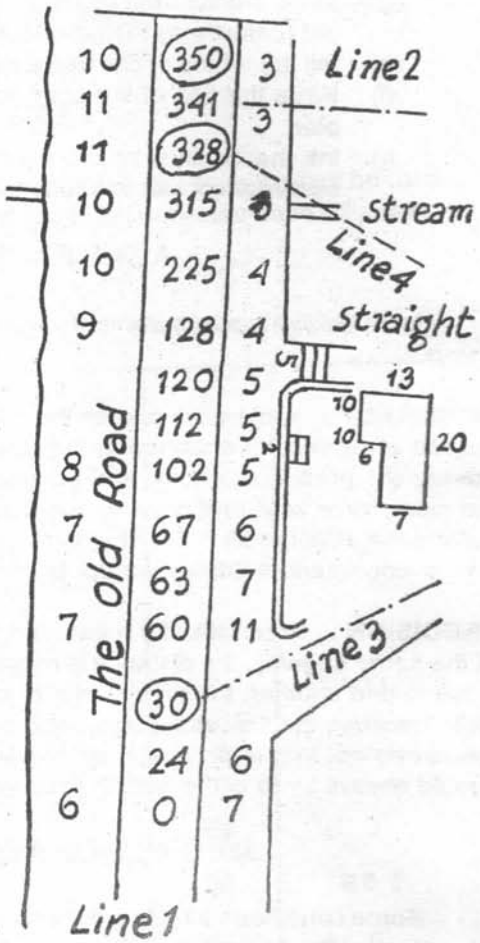
8. Details of any other line meeting at the starting point of the survey line.

PLOTTING A CHAIN SURVEY

Plotting means representing on paper to a suitable scale the already surveyed area. It consists in drawing plans and sections using conventional signs.

Instruments Required for plotting

(i) A drawing board and T-square.
(ii) Drawing box.
(iii) A set of set squares.
(iv) A set of scales.
(v) Proportionate compass.
(vi) A set of french-curves.
(vii) A set of offset scales.
(viii) Drawing paper - good quality and well seasoned.
(ix) Pencil, eraser, drawing pins, colour box, ink, knife, brushes, etc.



Double Line Field Book

Procedure of Plotting a Chair Survey : The plotting of a chain survey should be done in the following steps -

(a) Leave a margin 2 to 5 cm all round the drawing sheet.
(b) Choose a suitable scale so as to enable the whole area to be plotted within the sheet.
(c) Keep some space on the top for writting the title of survey and the scale used, and at bottom for writing the name of the party, date of survey.
(d) Plot the skeleton framework of triangles selecting the longest line as a base line.
(e) Check the accuracy of the plotted framework by measuring from the plan, some of tie lines or check lines and comparing their lengths with the measurements actually made in the field.

(f) Draw the north line at the top left hand corner.
(g) Draw offsets both on left and right hand of the survey lines and represent different features on the ground by different symbols.
(h) Fill the whole of the framework with details.
(i) Write the title of the plan, scale used and other requisites to be shown on the plan.
(j) Ink the outlines of the survey work, give a colour wash to various objects in accordance with the conventional signs, symbols and colours.

ACCURACY AND PRECISION

In dealing with measurements, it is important to distinguish between accuracy and precision.

ACCURACY : Accuracy denotes the closeness of a measurement to its true value. It is the degree of perfection obtained in measurement. Accuracy depends on (a) precise instruments, (b) precise methods, and (c) good planning. The use of precise instruments simplify the work, save time and provide economics. The use of precise methods eliminate or try to reduce the effect of all types of errors. Good planning includes proper choice of instruments and arrangement of survey control to save time and to reduce the possibility of errors.

PRECISION : Precision of a measurement denotes its closeness to antother measurement of the same quantity. If a distance is measured several times and the value obtained are very close to one another, the precision is high. But it does not necessarily mean that accuracy is high, because the values though close to one another may not be close to the true value. A measurement that is precise may also be accurate if it contains no error. A good surveyor should always try to achieve both precision and accuracy.

DEGREE OF ACCURACY IN CHAINING

Some conditions affecting the accuracy are (i) fineness of the graduations of the chain, (ii) nature of the ground, (iii) time and money available, (iv) weather conditions. The error may be expressed as $\frac{1}{n}$ which means that there is error 1 unit in the measured distance of n units.

Limits of Error : In mining, as in most other branches of surveying, it is usually necessary to establish a standard of accuracy. Safety of a mine and its workers is dependent on the accuracy of the plans and surveys of same mine and that of adjoining mines. Hence, standard of accuracy below which the results of surveys must not fall has been established after a long experience. The standards of accuracy established for each operation, e.g. traversing, angular measurements, levelling, correlation, etc. are referred as *Limits of Error.*

The following are the usual permissible limits of error in chaining under different conditions

1. For measurement with chain on rough or hilly ground..... 1 in 250

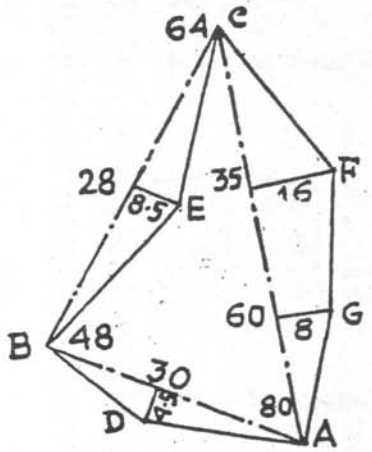
2. In ordinary chain survey with chain under average condition.... 1 in 500
3. For measurements made with tested chain, plumb bob, etc.... 1 in 1000

CALCULATION OF AREAS

The area enclosed by the chain line is split up into triangles whose areas can be calculated, and the area enclosed by chain lines, offsets and features, may also be readily calculated direct for usually the figures are simple geometrical ones, such as rectangles, triangles, trapeziums, etc.

Example 1. Draw a neat sketch of the field from the field notes in hectares.

Solution : The sides of the triangle ABC are 48m, 64m and 80m. Therefore, it is a right-angled triangle as its sides are in the ratio of 3: 4: 5.



Sketch of the field

Field Notes (given)

	A	
0	80	
G 8	60	
F 16	35	
	0	0
	C	Turn to right
	C	
	64	0
	28	8.5 E
	0	0
	B	Go NE
	B	
	48	
D 4.5	30	
	0	0
	A	Go NW

From the figure,

Area of triangle ABC = $^1/_2$ x 48 x 64
= 1536sq. m.

Area of triangle ABD = $^1/_2$ x 48 x 4.5
= 108 sq. m.

Area of triangle BCE = $^1/_2$ x 64 x 8.5 = 272sq. m.

Area of rectangle ACFG

$$= (^1/_2 \times 35 \times 16) + \frac{16 + 8}{2} (60 - 35) + ^1/_2 \times (80 - 60) \times 8 = 660 \text{ sq. m}$$

Total area of the field ADBECFG

= (1536 + 108 - 272 + 660) sq. metres

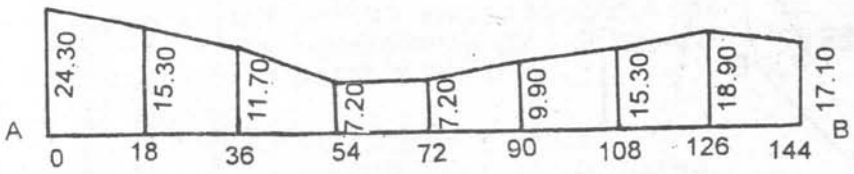
= 2032 sq. metres = **0.2032 Hectares.**

Example 2. Undernoted readings are from a record of chain survey. Calculate the area enclosed by it -

To	B
17.10 m	144 m
18.90 m	126 m
15.30 m	108 m
9.90 m	90 m
7.20 m	72 m
7.20 m	54 m
11.70 m	36 m
15.30 m	18 m
24.30 m	0 m
From	A

(Survey Comp. 1961)

Solution : A rough plotting is made starting from below upwards. Offset interval is 18 metres.



Area by Trapezoidal rule

$$= \frac{\text{Width}}{2} (\text{First offset} + 2 \times \text{Sum of intermediate offsets} + \text{Last offset})$$

$$= \frac{18}{2} [24.30 + 2(15.30 + 11.70 + 7.20 + 9.90 + 15.30 + 18.90) + 17.10].$$

$$= 9 (24.30 + 2 \times 85.50 + 17.10)$$

$$= 9 \times 212.40 = \mathbf{1911.60 \text{ sq. m}}$$

Example 3. The following perpendicular offsets were taken at 10m intervals from a survey line to an irregular boundary line :

9.8, 12.9, 17.5, 15.9, 19.8, 21.9, 24.5, 22.2 and 18.9. Calculate the area enclosed between the survey line, the irregular boundary line and the first and last offsets.

(Survey Comp. 1978)

Solution : Width of strip = 10 metres.

For even number of strips of equal width, the area of irregular figure can be calculated by **Simpson's Rule.**

$$\text{Area} = \frac{W}{3}(\text{first ordinate} + \text{last ordinate} + 2 \times \text{sum of all other odd ordinates} + 4 \times \text{sum of all even ordinates})$$

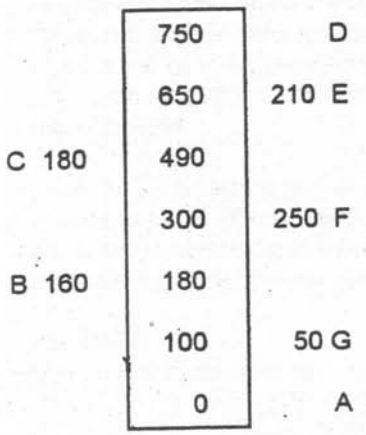
$$= \frac{10}{3}[9.8 + 18.9 + 2(17.6 + 19.8 + 24.5) + 4(12.9 + 15.9 + 21.9 + 22.2)]$$

$$= \frac{10}{3}[28.7 + 2 \times 61.9 + 4 \times 72.9] = \frac{10}{3} \times 444$$

$$= \mathbf{1480.33 \text{ sq. m.}}$$

EXERCISES

1. Plot the following survey of a field ABCDEFG and calculate its area.



(Ans. 21.4 hectares)

2. The following perpendicular offsets were taken at 15 metres intervals from the survey line to an irregular boundary line :

2.52, 3.12, 5. 46, 4.92, 6.09, 6.84, 7:68, 6.78 and 5.76 metres. Compute the area in square metres enclosed between survey line, the irregular boundary and the first and last offsets. **(Ans.** 666.9 sq. m.)

3. A chain line, length 60 metres, runs between the extreme ends of boundary fence, offsets are measured from equidistant points. The measurements are 0, 30.8, 50.6, 65, 66, 67.2, 64.50, 34.2, 20.9, 15, 8 and 0 metres. Find the area of land between chain line and the boundary fence. **(Ans.** 2302.9 sq. m.)

4. The following perpendicular offsets were taken at 10 metres intervals from the survey line to an irregular boundary line :

1.75, 5.60, 4.20, 6.65, 8.75, 6.20, 3.25, 4.20 and 4.25. Calculate the area enclosed between the survey line, boundary lines and the first and last offsets by the application of Simpson's Rule. **(Ans. 430 sq. m.)**

5. The following are measurements, in metres, taken along a chain line with offsets to the boundary fence of a field.

Chainage	0	50	100	140	200	124
Offsets	Fence	10.5	7.3	13.5	9.0	Fence

(a) Calculate from the measurements the area between the chain line and the boundary fence. (b) How many square centimetres would that area cover if plotted on a plan drawn to the scale of 1 : 1200 ? **(Ans. 1906.5 sq. m; 13.24 sq. cm.)**

CHAPTER - V

PLANE TABLE SURVEYING

★★★

The board can be mounted on a tripod stand by means of a central screw with a wing nut or in such a manner that the board can be revolved, levelled and clamped to any position. There are three types of tables according to different devices used for levelling the table - (i) Traverse table which is commonly used and levelled by adjusting tripod legs; (ii) Johnson table fitted with ball-and-shocket joint and a vertical spindle with two thumb screws for levelling and orientation; and (iii) Coast survey table fitted with three foot screws for levelling accurately generally used for work of high precision.

Plane tables are available in sizes 400 x 300 x 15mm, 750 x 600 x 20mm or 500 to 600mm square.

Alidade : It is a metal (usually of brass) or wooden straight edge 40cm to 60cm long, 3 to 5cm wide and fitted with two vanes at the ends. The working edge of the alidade is bevelled and graduated into cms and mms. This is known as *fiducial edge*. A line can be drawn against this edge when a line of sight passes through the object to be sighted from the instrument station. This plain alidade is not suitable on hilly area since the inclination of the line of sight is limited.

For taking inclined sights and where more accuracy is required a *telescopic alidade* is used. The telescope is fitted with a level tube, a vertical circle to measure vertical angle and also with cross and stadia hairs. Thus the observer can very quickly and easily obtain the true horizontal distance from the plane table to a levelling staff placed at the point and the difference in elevation between them.

Trough Compass : It consists of a narrow rectangular non-magnetic metallic box 8cm to 15cm long, 3cm to 5cm wide and 2cm to 3cm high covered with a glass cover. A magnetic needle is provided in the centre of the box. At the end of the compass graduated scales are attached with 0° at the centre and upto 5° on either side of the zero line. It is used for marking on paper the direction of magnetic meridian for orientation of the plane table to magnetic north.

Plumbing Fork : It consists of a hair-pin shaped light metal frame having arms of equal length, in which a plumb-bob can be suspended from a hook at the lower arm immediately below the end point of the upper arm. The plumbing fork is used for centering the table over the ground station.

WORKING OPERATIONS

It involves the following three steps :

1. Fixing : Fixing the table to the tripod.
2. Setting : (a) Centering.
   (b) Levelling the table.
   (c) Orientation.
3. Sightitg the points.

Centering : The operation of setting the table over the station on the ground so that the point plotted on the sheet is corresponding to the station occupied is called centering. This is done by means of a plumbing fork.

Levelling : For small scale work, the table is levelled by adjusting the legs. The levelling is done by keeping a spirit level on the board in two positions at right angles to each other and getting the bubble central in both positions adjusting the legs but without disturbing the centering. For more precise work, a Johnson table or Coast Survey table may be used.

Orientation : The process of setting the plane table in such a way at each of the successive stations so that all lines drawn on the paper should be parallel to the corresponding lines on the ground is called orientation. There are two main methods of orienting the plane table -

(a) Orientation by means of magnetic needle.
(b) Orientation by means of backsighting.

Magnetic Needle VS Backsight Methods of Orientation

Sl. N0.	Magnetic Needle Method	Back Sight Method
1.	Quick and less accurate.	Slow and accurate.
2.	Not suitable in magnetic areas	Suitable at all places.
3.	Most suited when no second point is available for orientation.	Only possible when the plotted position of the preceding station is known.
4.	Error, if any, in trough compass may lead to incorrect orientation thereby the error may accumulate.	No such possibility, because the trough compass is not used.
5.	There is no check on orientation.	The orientation can be checked by placing the trough compass along the meridian line drawn at start.

METHODS OF PLANE TABLE SURVEY

The methods of plane tabling are -

1. Radiation Method.
2. Traversing or progression Method.
3. Intersection Method.
4. Resection Method.
5. Radio Progression Method.

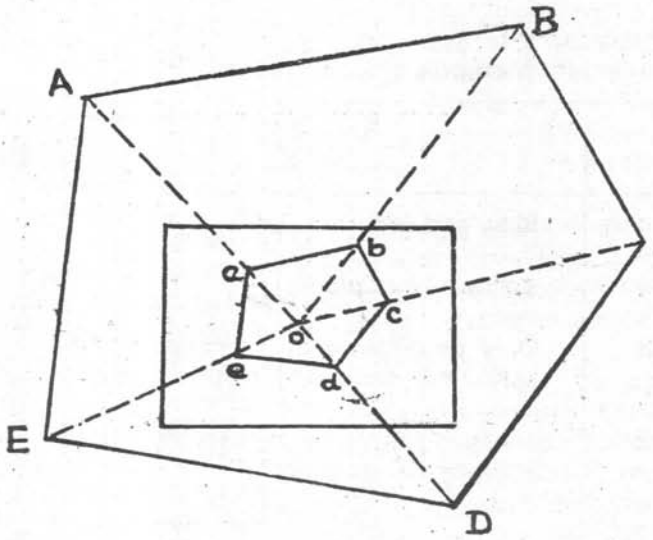
1. Radiation Method : When from a single set of plane table on instrument station different details are located on the sheet, the method is known as radiation.

2. Traversing or Progression Method : The method which consists in setting and shifting

the table at each station, orientation and measuring the lengths between stations is called traversing method.

3. Intersection Method : .When the location of an object is obtained on the sheet of paper by the intersection of the rays drawn after sighting at the object from two plane table stations previously plotted is called the intersection method.

4. Resection Method : The process which consists of determining the plotted position of the stations occupied by the plane table, by means of sights taken towards known points locations of which have been plotted is called Resection method.



Radiation method

5. Radio-Progression Method : This method is a combination of methods of radiation and traversing.

1. RADIATION

The table is set up and correctly levelled at any point from which all the stations to be surveyed are visible. A suitable point 'o' is marked on the paper to represent the instrument station. A pin is planted at o on the paper as a pivot for turning the alidade and sighting the stations to be surveyed. The alidade is now sighted to the various stations. A,B,C, etc. to be located on the paper and the radial lines towards them through the point o are drawn on the paper. The distace OA, OB, etc. are measured with a chain and these are set to scale on the paper as oa, ob, etc. By joining these points a, b, c, etc. by lines, a plot of the entire tract is obtained.

Advantages

1. For interior filling of a survey this method is very suitable.
2. It has a wide range if distances are measured by tacheometer.
3. Plotted length can be checked at once by measuring field distance.
4. The instrument dose not require to be shifted to other stations.

Disadvantages

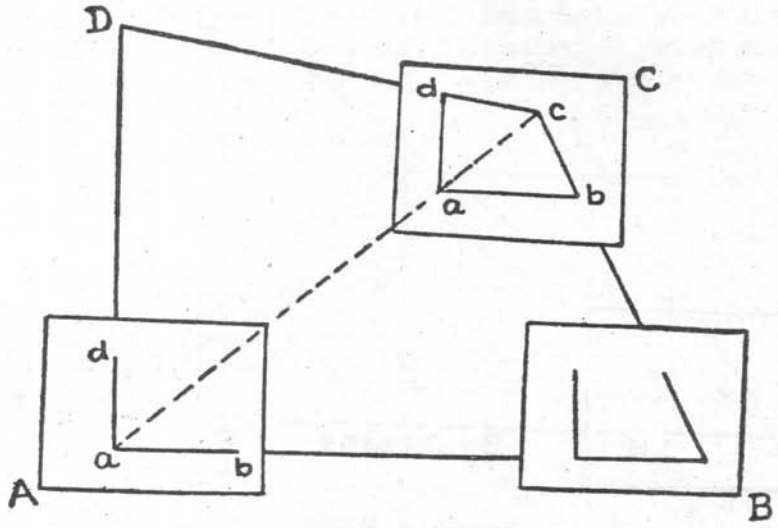
1. All the stations should be accessible.

2. TRAVERSING

Stations A, B, C, D, etc. are selected on the field for traverse. The table is set up and levelled over the station A. The station A is transferred on the paper at a. The board is clamped. The magnetic meridian is now drawn with the aid of trough compass.

With the alidade touching the point a, the station B is sighted and the measured distance of AB is scaled off on the paper as ab with a fine pencil line. The table is shifted to station B and set up and levelled with b exactly over B. The wing nut is loosened and the table is oriented to sight station A correctly with the alidade along the line ba on paper. The table is now clamped and the alidade is sighted to C with its edge touching the point b. A fine line bc is drawn to scale to represent the measured length BC.

The work should be proceeded in this way by orienting the table by a backsight before a foresight. Intermediate checks should be taken wherever possible.



Traversing Method

Advantages

1. This may be used both for closed or unclosed traverse line.
2. Details may be plotted by taking offsets.
3. Closing error may be determined.
4. Intermediate checks may be taken.
5. It is analogous to compass traversing when each orientation is made entirely by trough compass.

Disadvantages

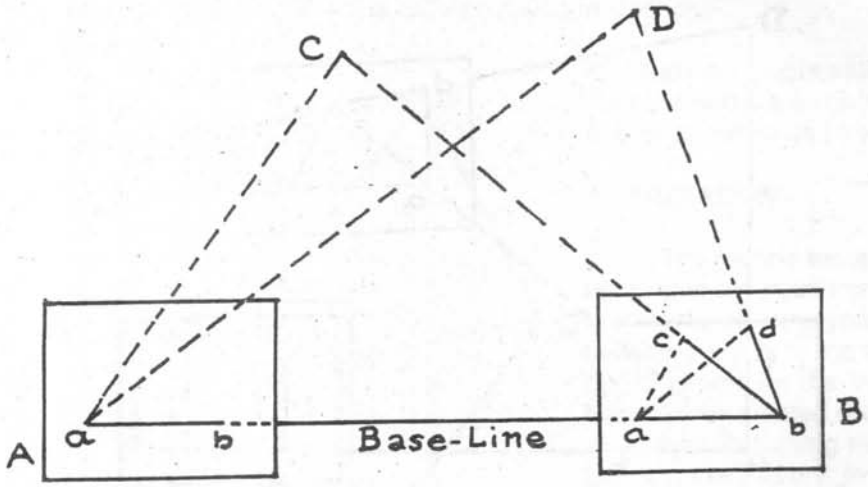
1. All the stations should be accessible.
2. The method is tedious, as it requires the instruments to be shifted to each station and orientation for backsight and foresight at each station occupied.

3. INTERSECTION

This method is also known as *triangulation*. The table is set up and levelled at A and the point is transferred on the paper at some convenient position a. The alidade is sighted to B with its edge touching the point a and the measured length of AB is plotted to scale on the

paper with a fine pencil line ab. The table is clamped in this position. The alidade touching the point a is now sighted to the stations C, D, etc. and fine rays are drawn from 'a' towards each.

The table is shifted to station B. It is set up and levelled with b exactly over B. The table is correctly oriented by backsighting to A. Rays are drawn through b towards the points previously sighted. Each point is thus located by the intersection of two rays towards it from the stations A and B. Now rays are drawn from B towards other points not sighted from A and then proceeded to C. In this way all the stations of the area to be surveyed are located.



Intersection Method

Advantages

1. The method is used for plotting details.
2. Only one linear measurement is required for the base line.
3. It does not require accessibility to all stations. The base line may be within or beyond the points to be plotted.
4. The method is simple, expeditious and the work can be cheaply executed.

Disadvantages

1. No station can be correctly located unless it has been intersected from at least three other stations.
2. Accuracy can not be expected unless the intersecting triangles are well-conditioned for yielding definite intersection and the position of station points are correctly transferred on paper.

4. RESECTION

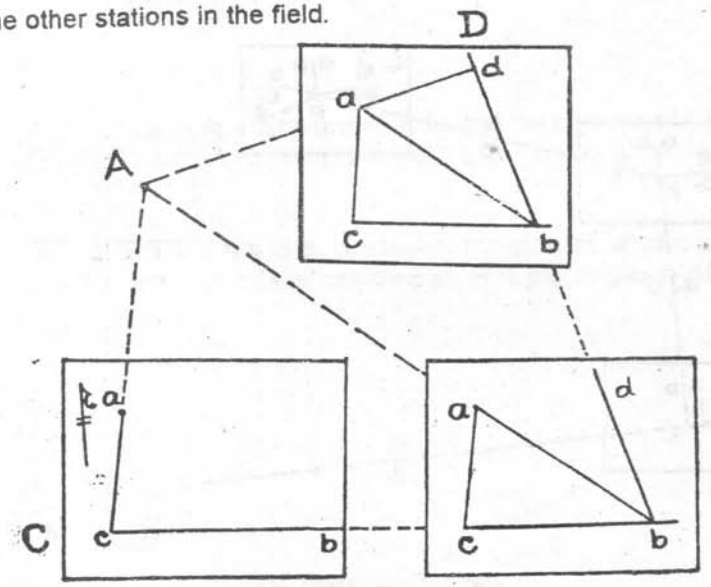
Let the stations A, B, C, D, etc are to be located on the paper.

The table is set up at C, taking 'c' on the paper to represent C on the ground and a line

is drawn on paper at some convenient position showing the magnetic meridian using the trough campass. The sihgts are directed to A and B and rays are drawn representing CA and CB along the alidade. The length of CA is measured and plotted to scale as ca on paper.

The table is moved on to B and placing the alidade along bc, the table is turned round till the point C is bisected by the sights. The table is clamped. The alidade is now placed against the pin at 'a' the sights are directed on A, and a line is drawn along the alidade which intersects bc. The point of intersection of these lines fixed the point B on the paper at b. Again, the station D is bisected and a line bd is drawn. The table is now removed to D and the station B is sighted back along the line db. The alidade is placed against the pin at 'a' and the station A is bisected or against the pin at 'c' the station C is bisected. The lines Aad and Ccd are drawn. The point of intersection of these lines with bd is the position of the station D on the paper at 'd'.

So on for the other stations in the field.



Resection Method

Advantages

1. The Only one linear measurement is required.
2. The accuracy of plotting is not much affected as it is confined to scale work.

Disadvantages

1. This method is used for location of station points only.
2. All the stations should be accessible.

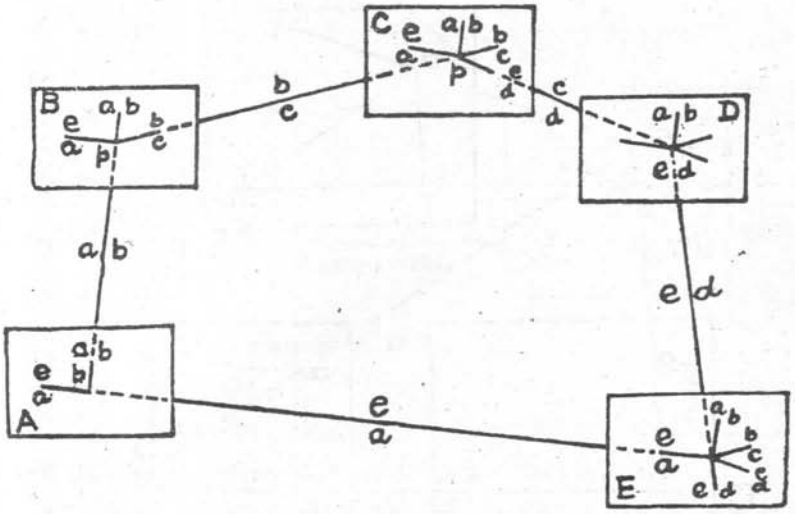
5. RADIO-PROGRESSION

This method is combination of the methods of radiation and traversing.
The figure represents a tract ABCDE, to be plotted by this method.

The table is set up at A, with its centre. The centre of the table being indicated by p, the line p (ea) and p (ab) are next drawn in the directions, EA and AB and equal to scale of those lines. Moving the instrument to B, it is set up with its centre over that point, the lines already drawn are brought parallet to corresponding ones on the ground, and the line p (bc) is drawn in a similar manner. The same procedure is followed on occupying each station.

All the stations having been occupied these will appear on the paper a series of lines radiating from the centre p and numbered (ab), (bc), (cd), etc. Then by starting at any point and drawiag a line ab, equal and parallel to p (ab), from b drawing a line bc, equal and parallel to p (bc), etc. a plot of the tract will be made.

The lines should be drawn on the directions from p to the lettered ends. As a final check the finished plot will close.



Radio Progression Method

Advantages

1. The difficulty in orientation of the table at each station as in traversing can be avoided.
2. A ready check is possible after completion of the plotting.

Disadvantages

1. It requires additional time for plotting oven after completion of traversing.

TWO POINT AND THREE POINT PROBLEMS

INTERPOLATION : Interpolation means eye-estimation. But in plane tabling it means determining the position of a point in the field over which the table is set up by taking observations

to at least three points, the positions of which are already plotted on the map.

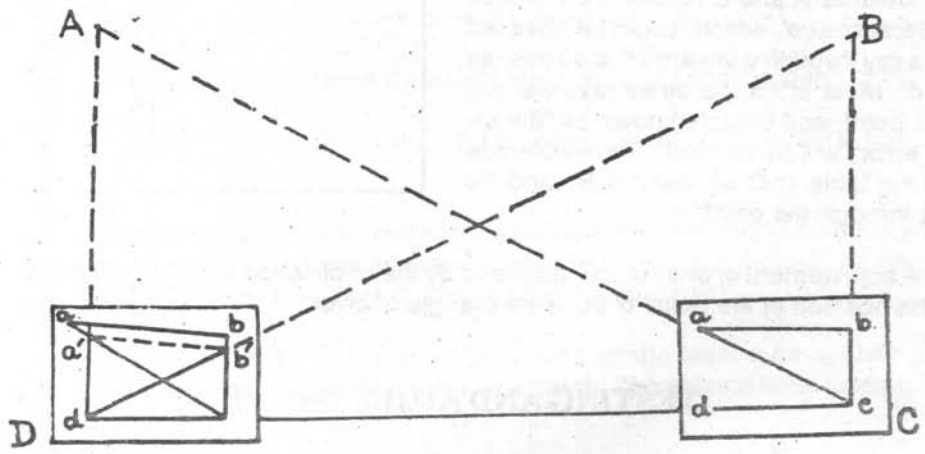
RESECTION : Sometimes it becomes necessary to locate one's position on the plan, during the course of plane tabling. This can be accomplished only when three stations are visible. This method of location in plane tabling is called resection.

TRIANGLE OF ERROR : The location of a point on the traverse of a plane-table survey is accomplished by means of two or three intersecting rays drawn from two or three stations depending on the importance. If the orientation of the table has been perfect the intersection of the rays is at the same point. Very often the third ray when drawn gives rise to a small triangle known as the *triangle of error* in respect of which the point is to be located. This triangle of error may be great or small depending upon how much or how little the table is out of azimuth.

TWO-POINT PROBLEM

The occasion may arise where it is desirable to locate the position of a point not plotted on the plan, from which only two plotted points are visible. Such a problem in plane tabling is known as Two-point problem.

Let the two visible points are A and B, plotted on the plan as 'a' and 'b'. C is the point which is to be located. A fourth point D is selected so that the angles CAD and CBD are betwen 30° and 120°.



The table is setup approximately in position at D, in estimation or by compass, and the lines Aad and Bbd are drawn to intersect at d. Through the point 'd' a line is drawn sighting the alidade towards C.

The table is next set up at C, and assuming the point 'c' on the line dC, at an estimated distance from d, the table is oriented by placing the alidade along cd and sighting D. Now the

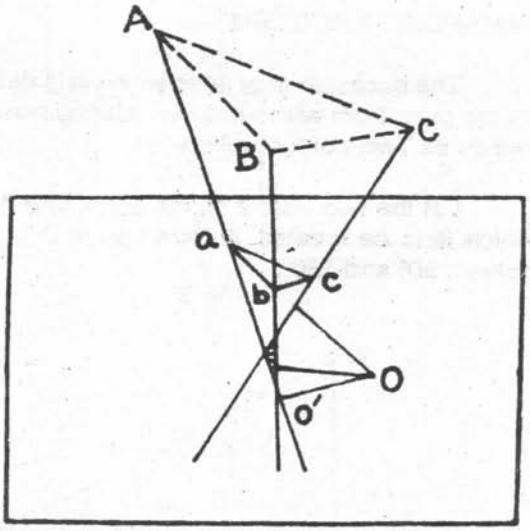
stations A and B are sighted with the alidade at c and the lines from c to A and from c to B are drawn intersecting da and db at points a' and b', which form a quadrilateral with cd similar to the true one but erroneous in size and position. The angle which the lines, ab and a'b', made with each other is the error in position.

By drawing through 'c' a line, cd', making the same angle with cd as that which ab made with a'b', and directing this line cd' to D, the table is brought into position, and the true point c is found by the intersections, aA and bB.

THREE-POINT PROBLEM

It involves for determining the position of an unknown point on the plan, during the course of a plane-table survey, when the point is visible from three stations already fixed on the plan.

Let the three stations A, B and C, whose plotted positions are a, b, c, are visible from a station O, whose position on the paper is to be found out.



The table is set up and oriented at O, the point whose position is required, so that ab and bc are parallel to AB and BC respectively. Two rays are drawn through a and b with the alidade directed towards A and B respectively. These would intersect at o', which could be checked whether a ray through c towards C also passes throung o'. Most often the three rays will not meet at a point, and triangle known as "the triangle of error" will be formed. The problem is to orient the table at O so that Aa, Bb and Cc will pass through the point 'o'.

The adjustement of orientation facilitated by the application of Lehman's rules for determining the position of the point 'o' from the triangle of error.

TESTING AND ADJUSTMENT

PLANE TABLE : The following tests and adjustments may be done in plane table to make it trustworty -

1. The top surface of the plane table must be perfect plane.

Test : It may be tested by moving a straight edge on the top surface in several directions.

Adjustment : If the surface is not perfectly plane and gives light through the edge touching the

table, remove high spots by planing or by rubbing with sand-papers.

2. The surface of the board should be perpendicular to the vertical axis of the instrument .

Test : Level the plane table after fixing it on the tripod. Place a spirit level on the table and centre the bubble. Fix in this position the legs of the table. Now turn the table through 180° and again see whether the bubble remains central.

Adjustment : If not, correct half the error by inserting a packing between the underside of the board and support. Level again. Repeat the process till adjustment is perfect.

ALIDADE : The following tests and adjustments are required to be made -

1. The fiducial edge (working edge) of the alidade should be straight .

Test : Draw a fine line by placing the alidade on a sheet of paper fixed on the table along the fiducial edge. Reverse the alidade end for end and put it against the ends of a line. Again draw a line, if two lines coincide the edge is straight.

Adjustment : If not, correct the edge by filing till the edge becomes straight.

2. The vanes of the alidade should be perpendicular to the base.

Test : Suspend a plumb bob at a short distance from the plane table. Place the alidade on the levelled table and sight the plumb line through the vanes. If the hair of the sighting slit coincides with the plumb line, the vanes are correctly fitted.

Adjustment : If not, adjust the vanes by inserting packing or filing the base of the sights.

3. The axis of the spirit level (if mounted) of the alidade should be parallel to the base of the alidade.

Test : Draw a line along the working edge of the alidade after levelling it on the plane table by means of attached spirit level. Now reverse it end for end and again place it along the previously drawn line. If the bubble is central, the adjustment is correct.

Adjustment : If not, correct half the error by means of the level tube adjusting screws if attached. Repeat the process till bubble remains central. The second level tube attached may also be tested and adjusted in a similar manner.

4. *Adjustment of Telescopic Alidade*

The following adjustments are needed in this case :

(i) The line of collimation should be parallel to the axis of the telescope.
(ii) The axis of the telescope should be parallel to the base of the alidade.
(iii) The axis of the telescope level must be parallel to the line of collimation.

(iv) The vertical circle should read zero when the line of sight is horizontal.
(v) The line of collimation should be perpendicular to the horizontal axis of the telescope.

ADVANTAGES AND DISADVANTAGES OF PLANE TABLING

The follwing are the advantages and disadvantages of plane table suveying -

ADVANTAGES

1. It is most suitbale for preparing small maps.
2. Omitted measurements can not occur since the plotting is done in the field.
3. Errors of measurements or plotting may be easily detected by check lines.
4. Various features are directly plotted on the paper, any errors liable to arise can be eliminated or any omissions can be easily detected and corrected on the spot.
5. No field-book is necessary, hence mistakes in booking the field-notes are avoided.
6. Most rapid method of surveying.
7. It is less costly than a theodolite survey.
8. Very suitable for magnetic area where compass survey in not reliable.
9. By the aid of skillful surveyor a complete method of triangulation can be adopted producing results of an accuracy almost equal to that obtained with good theodolites; its horizontal angles being fixed by rays aided by the telescopic alidade and independent of magnetic variations.
10. Mistakes in making and recording angular observations with the theodolite or compass and plotting them by an imperfect protractor - all such errors are next door to impossible with the plane-table survey.
11. In combination with a dumpy level it is very useful for contouring in moderately hilly terrain.
12. Little office work required.
13. No great skill is required to produce a satisfactory map.

DISADVANTAGES

1. It is impracticable in densely wooded country.
2. It can not be used in wet climate and is awkward in stormy weather.
3. The plotting can not be done when the sun is too hot or during night.
4. The plane-table being of limited size often changing of paper is necessary to carry

on the survey.

5. It is not suitbale for large scale surveys, and when surveying along a road or a river or in route surveying of any extent.
6. It is heavy, cumbersome and awkward to carry.
7. There are several accessories to be carried which are likely to be lost.
8. The table is likely to warp due to contraction.
9. Replotting to a different scale is not possible for want of field notes.
10. It is difficult to check the accuracy of plotting and closing error can not be detected.

ERRORS IN PLANE TABLING

SOURCES OF ERRORS : The following are Chief sources of error in plane-table surveying -

1. **Instrumental errors :** These are due to -

   (a) The surface of the table is warped or uneven and not a perfect plane.
   (b) The edge of the ruler is not a straight line.
   (c) The surface of the table is not perpendicular to the vertical axis.
   (d) The axis of the spirit bubble is not parallel to the base of the alidade.
   (e) The sight vanes of alidade are not perpendicular to its base.
   (f) Defective construction of the compass.

   If a telescope is used for long distances, error may arise due to its non-adjustments.

2. **Errors of manipulation and sights :** These are due to -

   (a) No-horizontal of the table.
   (b) Inaccurate centering.
   (c) Defective orientation of the table.
   (d) Insufficient clamping for which the table is rotating between sights.
   (e) Inaccurate sighting.

3. **Errors of plotting :** These are due to -
   (a) Rays not accurate by drawn through the station point.
   (b) Inaccuracy in plotting.

4. Errors due to shrinkage and warping of the paper.

PRECATUIONS REQUIRED : The following precautions are necessary to ensure accuracy when using plane table -

1. Necessary tests and adjustments of the plane table and alidade should be made prior to use.
2. The table should be set up firmly upon the tripod. The fittings of the table and stand should not be left loose.
3. The table should be perfectly levelled.
4. Its setting and orientation should be perfect.
5. It should be very exact with its setting in meridian.
6. Orientation of the table at a station should be done by sighting in the back station as it is not safe to depend entirely on the compass.
7. To avoid accumulation of errors the preceding station should be sighted from every third or fourth station.
8. The distance between any two stations, not consecutive, should be measured on the ground, or obtained by the stadia method and compared with plotted distance on the plan for check.
9. A pin planted at the station point should be used as a pivot for turning the alidade and sighting the next station.
10. All measurements and plotting must he correct.
11. Rays should be drawn with utmost fineness for which a finely pointed pencil should be used, for marking sight lines a hard pencil and for the delineation of actual features a softer pencil should be used.
12. Shrinkage and warping of the paper should be guarded against.

Example 1. In setting up the plane table at a station A, it was found that the point 'a', representing the station A on the plan was not exactly above the corresponding station A on the ground. If the displacement of 'a' in a direction at right angles to a ray to P (AP) was 20cm. find the consequent displacement of 'p' from its true position if -

(a) Scale of the plan, 1 cm = 100m; AP = 1600m.

(b) Scale of the plan (R.F.) = $^1/_{400}$; AP = 32m.

(c) Scale of the plan, 1cm = 2m; AP = 16m.

Solution : (a) Displacement of p from its true position = ap x error in direction.

$$\text{Now, } ap = \frac{1600}{100} \text{ cm in direction } \frac{0.20}{1600} \text{ radian.}$$

$\therefore$ Displacement of p from its true position = $\frac{1600}{100}$ cm x $\frac{0.20}{1600}$ = **0.002cm.**

(b) Displacement of p from its true position = $\frac{32 \times 100}{400}$ x $\frac{0.20}{1600}$ = **0.05cm.**

(c) Displacement of p from its true position = $\frac{16}{2}$ x $\frac{0.2}{16}$ = **0.1cm .**

Example 2. (a) Enumerate the different methods of pin pointing the position of plane table from observations taken to three known stations.

(b) The rectangular co-ordinates (in metres) of three trigonometrical stations A, B and C, reduced to local origin at A, are -

A	0	0
B	16.20 N	114 E
C	54 S	174 E

The angle measured from a station P are -

$\angle APB = 58^0$ and $\angle BPC = 45^0$. Find out the co-ordinates of station P by any method.

(Survey Comp. 1961)

Solution : (a) The following methods are adopted to pin point the position of plane table from observations taken to three known stations :

(i) Mechanical Method of Tracing Cloth or Paper Method.

(ii) Graphical Method or Bessel's Method.

(iii) Trial and Error or Triangle of Error Method applying Lehman's Rule.

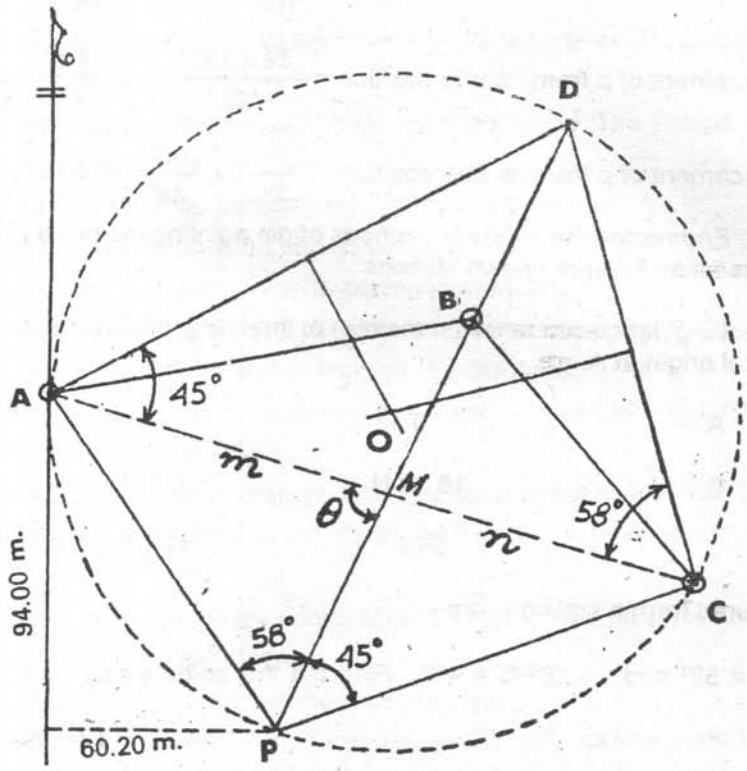
(b) METHOD (BY PLOTTING) :

The co-ordinate positions of stations A, B and C are plotted on a scale 1cm = 20m.

Angles ACD = 58° and CAD 45° are set out. The perpendicular bisectors of AD and CD are intersected at a point O. A circle is drawn through the points A, C and by taking O as centre of the circle.

DB is joined and it is produced to meet the circumference of the circle at P. P is the position of the observing station.

Angles APD and ACD are on the same arc AD subtending angles at the circumference of the circle. Therefore $\angle APD = \angle ACD$. Similarly angles DPC = DAC.



Co-ordinates of the observing stations P are measured off with the same scale 1cm = 20m. The co-ordinates of P in metres are :

Latitude = 94.00 S
Departure = 60.20 E

EXERCISES

1. Describe briefly the advantages and disadvantages of plane tabling ? What is two point problem in plane tabling ? How is it solved ? **(A.M.I.E. 1978)**

2. What are the advantages and disadvantages of plane table surveying ? What are the accessories and personnels required in a plane table surveying ? Given three points A, B, C visible from a point P over which the table is set, and a, b and c are their plotted positions. Describe a method of plotting 'p' if the point p is situated within the triangle ABC. **(Survey Comp. 1976, 1980)**

3. What are the main sources of errors in plane tabling ? How can they be kept to the mini-

mum ? How can you test and adjust an alidade ?

4. Describe various methods of plane tabling briefly. What are the advantages of telescopic alidade ?

5. What do you understand by three point problem ? Describe in detail the method of solving three point problem in the field. **(A.M.I.E. 1969)**

6. In setting up the plane table at a station P the corresponding point on the plan was not accurately centred above P. If the displacement of P was 20cm in a direction at right angles to the ray, how much on the plan would be the consequent displacement of a point from its true position, if (i) scale 1cm = 100m and (ii) scale 1cm = 2 metres ?

(Ans. 0.03mm; 1.5mm**)**

CHAPTER - VI

DIAL SURVEYING

★★★

which are connected but not return to the starting point or close upon a point of equal or greater order of accuracy. Open traverses are sometimes used on route surveys, but generally should be avoided because they offer no means of checking for errors and mistakes. In open traverses, measurements must be repeated to guard against mistakes.

THE COMPASS : The compass has been used by navigators and others for many centuries to determine directions. Prior to invention of the transit and sextent, the compass furnished surveyors with only practical way to measure directions and horizontal angles.

A compass consists of magnetised steel needle mounted on a pivot at the centre of a graduated circle. The needle points towards *magnetic north*. The north and south magnetic poles are located approximately 1600km and 2500km, respectively, from the true geographical poles. The magnetic lines of the earth, which align the needle, pull or dip one end of the needle below a horizontal position. The *angle of dip* varies from 0° at the equator to 90° at the magnetic poles. In the northern hemisphere, to balance the effect of dip and keep the needle horizontal, the south end is weighted with a very small coil of wire. As the compass box is turned, the needle continues to point toward magnetic north and gives a reading which is dependent upon the position of the graduated circle.

ANGLES, BEARINGS AND AZIMUTHS

ANGLES : An angle is defined as the amount of turning of a line revolving about one of its extermities, in one plane, from one position to another. Angle measured in surveying are classified either as *horizontal* or *vertical*, depending upon the plane in which they are measured. Horizontal angles are the basic measurements needed for detemining bearings and azimuths. The magnitude of an angle is expressed in sexagesimal system in india, i.e. degrees, minutes and seconds. The kind of horizontal angles commonly used in surveying are (1) *Interior and Exterior angles*, (2) *Clockwise and Counterclockwise angles*, and (3) Deflection angles. Deflection angles are measured either clockwise (right) or counter clockwise (left) from the extension of the back line to the forward station are always less than 180°.

A circle contains 360° of four right-angles and the four quarters into which a circle is divided by two diameters of right angles are called *Quadrants*. In surveying the 0° - 180° is the North-South line (or meridian), and the 90° - 270° line is the east-west line. The first or north-east quadrant includes all angles between 0° and 90° , the second or south east quadrant includes all angles between 90° and 180° , the third or south-west quadrant between 180° and 270° , and the fourth or north-west quadrant between 270° and 360°

BEARINGS : These represent one system of designating directions of lines. The bearing of a line is the horizontal angle subtended between the line concerned and some standard line of reference, termed a meridian. The meridian may be either (1) the geographical or true meridian, (2) the magnetic meridian, (3) an artificial meridian or (4) an assumed (or false) meridian. The bearings, may be stated either as (a) *Quadrant bearings*, measured from either end of the meridian according to circumstances, or (b) *Whole-circle bearings*, measured from one end of the meridian.

AZIMUTH : The azimuth of a line is its whole-circle bearing measured clockwise from the North or zero end of the meridian. The value of such bearing ranges from 0° to 360° . The bearing observed with a prismatic compass are the whole-circle bearing. The cardinal points of the compass are North, South, East and West, and their azimuth are North (0° or 360°), East (90°), South (180°) and West (270°) .

Quadrant Bearing : The quadrant bearing of a line is the angle between the line and the meridian as measured from its North or South end, whichever is nearer, and is always less than one right-angle. Each quadrant is graduated 0° to 90°, the zero being N and S points. It is, therefore, essential to mention the quadrant in which the line lies. The bearing of the line is commonly expressed by writing the numerical value of bearing between the letters N - E, or S - E, or S - W, or N - W as the case may be. This is also called *reduced bearings*. The bearing obsered with a Surveyor's Compass or Dial are the quadrant bearings.

CONVERSION OF BEARINGS : The following table referred to for conversion of whole circle bearings (W. C. B) to quadrant (or reduced) bearings and reduced bearings (R. B.) into whole circle bearings :

Table 1

Conversion of W. C. B. into R. B.

Case	W.C.B. between	Rules for R. B.	Quadrant
I	0° and 90°	R. B. = W. C. B.	N E
II	90° and 180°	R. B = 180° - W. C. B.	S E
III	180° and 270°	R. B. = W. C. B. - 180°	S W
IV	270° and 360°	R. B. = 360° - W. C. B.	N W

Table 2

Conversion of R. B. into W. C. B.

Case	R. B. in quadrant	Rule for W. C. B.	W. C. B. Between
I	N E	W. C. B. = R. B.	0° and 90°
II	S E	W. C. B. = 180° - R. B.	90° and 180°
III	S W	W. C. B. = R. B. + 180°	180° and 270°
IV	N W.	W. C. B. = 360° - R. B.	270° and 360°

FORE AND BACK BEARINGS

Every line has two bearings one observed at each end of the line. If the bearing of line AB is measured from A towards B, it is known as fore bearing and if the bearing of the line is

measured from B towards A, it is known as back bearing or reverse bearing. The back bearing of the line AB is, therefore, the fore bearing of BA. The fore and back bearings of a line will always differ by exactly 180°. In quadrantal system the fore and back bearings of a line are numerically equal but with opposite letters.

In Whole-circle system -

Back bearing = Fore bearing $\pm$ 180°.
Use plus (+ve) sign when the F.B. is less than 180° and minus (- ve) sign if exceeds 180°

In Quadrantal system -

If the fore bearing AB = N 35° E
Back bearing = S 35° W = Fore bearing of BA.

MERIDIANS : The meridian of chosen line of reference from which bearings are measured may be -

1. Geographical or True Meridian : It is the imaginary vertical plane (represented on a plan by a line) which passes through the point of observation and also through the North and South Geographical poles. As the earth is approximately spherical, meridians are not paralle! to one other, these converge from the equator to the poles. But for small surveys they are considered to be parallel.

The direction of the true meridian at a given place is always constant. The bearing of any line in relation to this meridian is called a *true bearing* or a geographical bearing, and it is a constant quantity.

2. Magnetic Meridian : It is the imaginary vertical plane which passes through the centre-line of a magnetic needle freely suspended from its centre at the point of observation, and influenced only by the earth's magnetic attraction. Its direction is not constant, but varies from moment to moment owing to changes in the earth's magnetic field. The bearing of a line in relation to the magnetic meridian is called a *magnetic bearing*.

3. Artificial Meridian : An artificial meridian is one laid down between two permanent objects, and is sometimes used when surveying an isolated area. Usually, the relationship of this line to geographical north is found as soon as convenient, but this is not essential so far as the correct relationship of alll the lines in the isolated survey is concerned.

4. Assumed of False Meridian : It may be any line in a survey, but usually the first draft. The bearings of all the remaining lines are obtained in relation to this assumed meridian and may subsequently be converted to geographical or magnetic bearing if the true or magnetic bearing of any one line is known.

Example 1. Convert the following whole-circle bearings into quadrant bearings :

(i) 69°30' (ii) 125°40' (iii) 241°10' (iv) 330°20'20"

Solution : (i) Reduced bearing = **N 69°30' E**

(ii) Reduced bearing = 180° – 125°40' = **S 54°20' E**

(iii) Reduced bearing = 241°10' – 180° = **S 61°10' W**

(iv) Reduced bearing = 360° – 330°20'20" = **N 29°39'40" W**

Example 2. Convert the following reduced bearings to whole-circle bearings :

(i) N 35° E (ii) S 30°30' E (iii) S 44°40' W (iv) N 58°10' W.

Solution : (i) Whole-circle bearing = **35°**

(ii) Whole-circle bearing = 180° – 30°30' = **149°30'**

(iii) Whole-circle bearing = 180° + 44°40' = **224°40'**

(iv) Whole-circle bearing = 360° – 58°10' = **301°50'**

Example 3. The following are the observed fore bearings of the lines :

(a) AB = 49° 50' (b) CD = 320°40' (c) PQ = N 38°30' E (d) RS = S 50°20' W.
Find out the back bearings of the lines.

Solution : Back bearing of a line in W.C.B. system = fore bearing ± 180°.

(a) B.B of line AB = 49°50' + 180° = **229° 50'**

(b) B.B of line CD = 320°40' – 180° = **140°40'**

Again, the back bearing of a line in quadrantal system is equal to its Fore bearing but with opposite letters. Hence -

(c) B.B of the line PQ = **S 38°30' W**

(d) B.B of the line RS = **N 50°20' E**

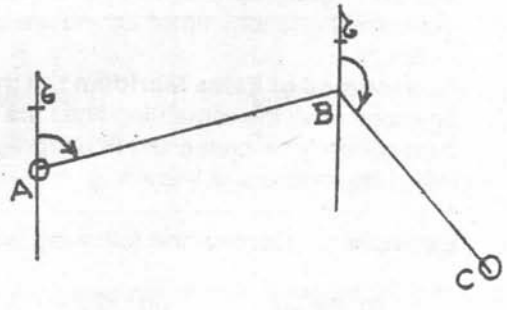
CALCULATION OF ANGLES FROM BEARINGS

case I. Given the bearings of lines in the whole-circle system :

(a) When the bearings of the two lines as measured from the point of intersection are given -

The included angle between them will be the difference of the two bearings.

(b) When the bearings of two consecutive lines are given but not from their point of intersection -



Express the given bearings as if they are taken from the common point of intersection.

The included angle ABC = B.B. of line of AB – F.B. of line BC.

Case II. Given the reduced Bearings of lines.

Rule (i) If the two lines are on the same side of the meridian and in the same quadrant, the included angle will be the difference of the two reduced bearings.

Rule (ii) If the two lines are on the same side of the meridian but in the different quadrants, the included angle in between them = 180° - the sum of their reduced bearings.

Rule (iii) If the two lines are not on the same side of the meridian but they are in the adjeacent quadrants, the included angle in between them = 180° – difference of the reduced berings of the two lines.

Rule (iv) If the two lines are not on the same side of the meridian but they are in the adjacent quadrants, the included angle = Some of their reduced bearings.

Example 4. (a) Find the angles between the lines AB and AC, if their respective bearings are :

(i) 35°40' and 142°20'
(ii) 130°20' and 252°45'

(b) The bearing of a line AB = N 85° E and the angle BAC = 92° clockwise. Find the bearing of AC.

Solution : (a) As the lines meet at common point A, therefore the included angle will be the difference of the two bearings.

(i) ∠BAC = bearing of line AC - bearing of line AB = 142°20' – 35°40' = **106°40'**

(ii) ∠BAC = 252°45' – 130°20' = **122° 25'.**

(b) Whole-circle bearing of line AB = 85°

Bearing of line AC = 92° + 85° = 177°

R. B. of line AC = 180° – 177° = **S 3° E**

Example 5. Find the angle between the lines AO and OB if their respective bearings are -

(i) 131°10' and 625°0' (ii) N 60°30' E and S 45°20' E

Solution : In such questions first convert the bearings of the lines OA and OB as if measured from the common point of intersection (O) and then find the angle between them.

(i) F.B. of the line AO = 131°10'
B.B. of the line AO = 131°10' + 180° = **311°10'**
or, F.B. of the line OA = 311°10'

F.B. of the line OB = 65° 20'

Hence, Angle AOB = 311° 10' – 65° 20' = **245° 50' (Exterior)**

(ii) R.B. of the line AO = N 60° 30' E

B.B. of the line AO =. S 60° 30' W

or, F.B. of the line OA = S 60° 30' W

F.B. of the line OB = S 45° 20' E

As the two lines are in the adjacent quadrants, -

The angle AOB = 60° 30' + 45° 20' = **105° 50'**

Example 6. Caculate the interior angles of traverse provided the bearings of the sides of the traverse ABCDE are given as follows -

Line	Fore Bearing	Back Bearing
AB	97° 15'	277° 15'
BC	12° 00'	192° 00'
CD	271° 30'	91° 30'
DE	189° 15'	9° 15'
EA	124° 45'	304° 45'

Solution : A rough sketch of the traverse may be drawn to facilitate in calculating angles.

B.B of line AB = 277° 15'

F.B of line BC = 12° 00'

Difference = 265° 15' Exterior angle ABC.

Interior angle B = 360° - 265° 15' = **94° 45'.**

B.B of line BC = 192° 00'

F.B of line CD = 271° 30' ∴ ∠C = 271° 30' - 192° 00' = **79° 30'**

B.B of line CD = 91° 30'

F.B of line DE = 189° 15' ∴ ∠D = 189° 15' - 91° 30' = **97° 45'**

B.B of line DE = 9° 15'

F.B of line EA = 124° 45' ∴ ∠E = 124° 45' - 9° 15' = **115° 30'**

B.B of line EA = 304° 45'

F.B of line AB = 97° 15' ∴ ∠A = 304° 45' - 97° 15' = 207° 30' **Ext.**

Interior angle A = 360° - 207° 30' = **152° 30'**

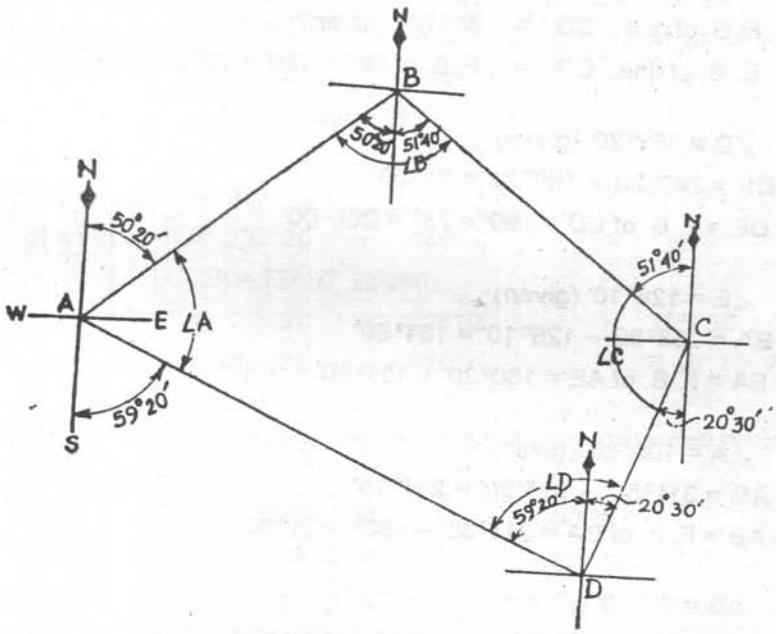
Check : The sum of the interior angles of the closed traverse with five sides must be equal

to (2 x 5 - 4) right angles = 6 x 90 = 540°. The sum of the angles = $\angle A + \angle B + \angle C + \angle D + \angle E$ = 152° 30' + 94° 45' + 79° 30' + 97° 45' + 115° 30' = 540° 00'.

Example 7. The following bearings were taken in running a travers ABCD :

Line	Reduced Fore Bearings
AB	N 50° 20' E
BC	S 51° 40' E
CD	S 20° 30' W
DA	N 59° 20' W

Calculate the interior angles and apply the usual check.



Solution : In case of quadrantal system the back bearings of the lines are numerically the same as their fore bearings but with opposite letters.

A rough sketch is drawn to facilitate calculations.

The value of $\angle A$ = 180° - (Sum of F. B. of lines AB and AD)
= 180° - (50° 20' + 59° 20') = **70° 20'**

$\angle B$ = Sum of the F. B. of lines BA and BC
= 50° 20' + 51° 40' = **102° 00'**

$\angle C$ = 180° - (Sum of F. B of lines CB and CD)
= 180° - (51° 40' + 20° 30') = **107° 50'**

$\angle D$ = Sum of F. B. of lines DC and DA
= 20° 30' + 59° 20' = **79° 50'**

Check : The Sum of interior angles should be equal to (2n - 4) right angles = $(2 \times 4 - 4) \times 90^\circ$ = 360°. Here n = 4.

$\angle A + \angle B + \angle C + \angle D = 70^\circ 20' + 102^\circ 00' + 107^\circ 50' + 79^\circ 50' = 360^\circ 00'$

Example 8. A closed traverse is run clockwise and the following interior angles were observed :

$\angle A = 100^\circ 20'$ $\angle C = 70^\circ 40'$ $\angle E = 126^\circ 10'$

$\angle B = 80^\circ 30'$ $\angle D = 162^\circ 20'$

If the fore bearing of the line CD = $60^\circ 20'$, find the bearings of all other lines.

Solution : F. B. of line CD = $60^\circ 20'$ (given)

B. B. of line CD = F. B. of DC = $180^\circ + 60^\circ 20' = 240^\circ 20'$

$\angle D = 162^\circ 20'$ (given)

F. B. of line **DE** = $240^\circ 20' - 162^\circ 20' = \mathbf{78^\circ 00'}$

B. B. of line DE = F. B. of ED = $180^\circ + 78^\circ = 258^\circ 00'$

$\angle E = 126^\circ 10'$ **(given)**

F. B. of line **EA** = $258^\circ 00' - 126^\circ 10' = \mathbf{131^\circ 50'}$

B. B. of line EA = F. B. of AE = $180^\circ 20' + 131^\circ 50' = 311^\circ 50'$

$\angle A = 100^\circ 20'$ (given)

F. B. of line **AB** = $311^\circ 50' - 100^\circ 20' = \mathbf{211^\circ 30'}$

B. B. of line AB = F. B. of BA = $211^\circ 30' - 180^\circ = 31^\circ 30'$

$\angle B = 80^\circ 30'$ (given)

F. B. of line **BC** = $360^\circ - (80^\circ 30' - 31^\circ 30') = 360^\circ - 49^\circ = \mathbf{311^\circ 00'}$

B. B. of line BC = F. B. of CB = $311^\circ - 180^\circ = 131^\circ 00'$

$\angle C = 70^\circ 40'$ (given)

F. B. of line **CD** = $131^\circ 00' - 70^\circ 40' = \mathbf{60^\circ 20'}$ (checked)

Example 9. A traverse was run anticlockwise and following interior angles were observed :

$\angle A = 72^\circ 10'$ $\angle C = 98^\circ 20'$

$\angle B = 100^\circ 30'$ $\angle D = 89^\circ 00'$

If the fore bearing of the line AB is S $50^\circ 30'$ W. find the bearings of all other sides in quardrantal system.

Solution : Fore Bearing of the line AB = **S 50° 30' W** (given)

Whole circle bearing of line AB = 180° + 50° 30' = 230° 30'

B. B of line AB = F. B. of BA = 230° 30' – 180° = 50° 30'

$\angle B = 100° 30'$ (given)

∴	F. B. of line BC	=	50° 30'	+	100° 30'	= 151° 00'
	R. B. of line BC	=	180°	–	151°	= **S 29° E**
	B. B. of line BC	=	151°	+	180°	= 331° = F. B. of line CB

$\angle C = 98° 20'$ (given)

	F. B. of line CD	=	331°	+	98° 20'	= 429° 20'
	R. B. of line CD	=	429° 20'	–	360°	= **N 69° 20' E**
	B. B. of lind CD	=	69° 20'	+	180°	= 249° 20'

$\angle D = 89° 00'$ (given)

∴	F. B. of line DA	=	249° 20'	+	89° 00'	= 338° 20'
	R. B. of line DA	=	360°	–	338° 20'	= **N 21°40' W**
	B. B. of line DA	=	338° 20'	–	180°	= 158° 20'

$\angle A = 72° 10'$ (given)

	F. B. of line AB	=	158° 20'	+	72° 10'	= 230° 30'
	R. B. of line AB	=	230° 30'	–	180°	= **S 50° 30' W**

Example 10. The following are the angles of a traverse measured in clockwise direction from the back station -

MAB	136° 14' 20"	60 m
ABC	172° 16' 40"	30 m
BCD	96° 37' 10"	40 m
CDE	217° 54' 30"	45 m
DEF	81° 10' 20"	73 m
EFG	168° 46' 50"	60 m

The bearing of the line MA is 327° 18' 40". What is the bearing of FG ?

(Survey Comp. 1962)

Solution : The bearings of the lines are calculated and tabulated below -

Angle	Value of Angle	Calculation		Bearing	Line
MAB	136° 14' 20"	327° 18' 40" + 136° 14' 20"	= 463° 33' 00"	327° 18' 40"	MA
		Less 180°	= 283° 33' 00"	N 76° 27'W	AB
ABC	172° 16' 40"	283° 33' 00" + 172° 16' 40"	= 455° 49' 40"		
		Less 180°	= 275° 49' 40"	N 84° 10' 20"W	BC
BCD	96° 37' 10"	275° 49' 40" + 96° 37' 10"	= 372° 26' 50"		
		Less 180°	=192° 26' 50"	S 12° 26' 50"W	CD
CDE	217° 54' 30"	192° 26' 50" + 217° 54' 30"	= 410° 21' 20"		
		Less 180°	= 230° 21' 20"	S 50° 21' 20"W	DE
DEF	81° 10' 20"	230° 21' 20" + 81° 10' 20"	= 311° 31' 40"		
		Less 180°	= 131° 31' 40"	S 48° 28' 20"E	EF
EFG	168° 46' 50"	131° 31' 40" + 168° 46' 50"	= 300° 18' 30"		
		Less 180°	= 120° 18' 30"	S 59° 41' 30"E	FG

Hence the Bearing of FG is **S 59° 41' 30"E.**

MAGNETIC DIP & DECLINATION

MAGNETIC DIP : If a magnetic needle is suspended freely at its centre of gravity, the needle will turn and point in the direction of the magnetic meridian and it will also dip downwards (except at the magnetic equator) at an angle to the horizontal plane. This vertical angle made by the magnetic force with the earth's surface is called the *dip*. The angle of dip is different for different positions on the earth's surface. In the northern hemisphere, the north end of the needle dips downwards, whereas in the southern hemisphere, the south end of the needle dips downwards. At the magnetic poles, the dip is 90°, the needle becomes vertical. As the lines of force are parallel to the surface of the earth at the magnetic equator, the needle remains horizontal and the dip is zero. At any other place on the surface of the earth, the dip varies from 0° to 90°, depending upon the lines of force at that point.

Isoclinec Line : At some points on the earth's surface there are a series of places having same magnetic dip and a line joining such points is called a *Isoclinic line.*

Aclinic Line : On the earth's surface between the magnetic poles there must be one point where the magnetic dip is zero. The locus of all such points is called the magnetic equator or *Aclinic Line.*

DECLINATION : The *declination* of the magnetic needle is the horizontal angle between the true and magnetic meridian at a given moment and place. An east declination is obtained if the magnetic meridian is east of true north, and west declination if it is west of the true north. The value of the declination at any particular location can be obtained (if there is no local attraction) by establishing a true meridian from astronomical observation or by a gyroscope and then reading the compass while sighting along the true meridian.

The angle of declination is not constant but varies continually. True meridian does not coincide with the magnetic meridian except very unfrequent intervals, in fact, it is 259 years since the two planes coincided each other.

Agonic Line and Isogonic Line : At some points on the earth's surface at a given time, the declination is *nil,* and a line joining such points of zero declination where the magnetic meridian coincides with the true meridian is called an *agonic line.*

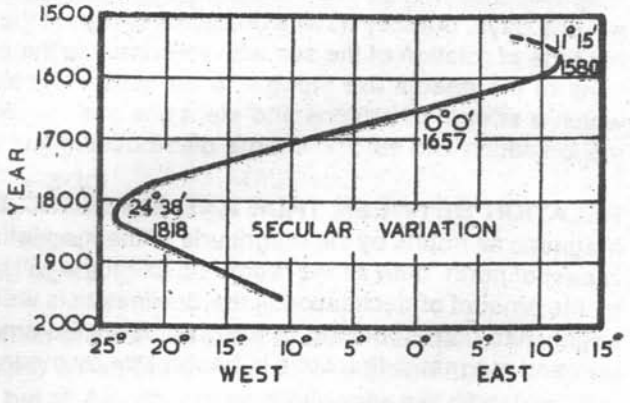
The magnetic meridians may diverge considerably at various angles to the east or west of true north, but there are naturally a series of points on the earth's surface which have similar declination. A line joining such point of equal declination at a given time is termed as *Isgonic line.* A map on which such lines are shown is called a declination map.

CHANGES IN THE MAGNETIC DECLINATION

The declination or deviation of the magnetic meridian from the true north is not constant either in time or place; it varies in value between different places at the same moment and from time to time in the same place. This continuous alteration is caused by the magnetic poles changing their position relative to the geographical poles, for which movement no satisfactory explanation has been given.

The changes in the amount of declination, known as variation, are of three types, namely (1) Secular, (2) Diurnal; and (3) Irregular variations.

1. Secular Variation : The magnetic needle, freely suspended from its centre, swings like a pendulum. It swings in one direction, say towards east, for a long term of years (100 to 150 years), gradually comes to rest and then swings in the opposite direction - towards west. It takes probably 2 to 3 centuries to complete a cycle. This phenomenon of long continued movement of a magnetic needle in one direction is observed all over the world. From the fact that it continues through so many years, this change in the direction of the needle is known as secular variation.



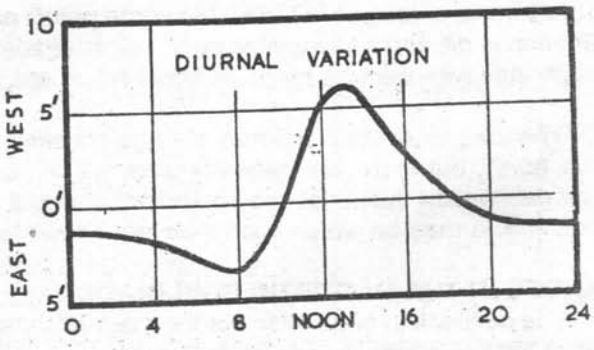
Secular Variation

Because of this enormous change in the value of the declination, a magnetic meridian shown on a plan must always be dated and due allowance should be made for change in the secular variation if additions are made to the plan in subsequent years.

Records which have been kept at London for four centuries show a range in magnetic declination from 11°15'E in 1580, to 24°38'W in 1818 back to 8°W in 1950, and 6°58'W in 1975.

2. Diurnal Variation : The daily or diurnal variation of the needle is the fluctuations of the needle from its mean position during a period of 24 hours i.e. in a day. The diurnal variation is believed to be caused by the effect of electrical currents in the upper atmosphire. The normal movement of the needle in a day is shown by a graph in the figure and by the followng approximate time table -

About 8 a. m. ....Maximum easterly.
10 a. m. .....Needle in mean position.
1. p. m. ....Maximum westerly.
6 p. m. .....Needle in mean position.



Diurnal Variation

The departure, easterly or westerly, from the mean value for the day is called *diurnal inequality*, and rarely exceeds 3 minutes on magnetic quiet days. On the highly disturbed days it may reach one degree or even more. The magnetec disturbances are generally most frequent during the night hours upto about 3 or 4 a. m. and they are fewest between 8 a. m. to 2 p.m. is recommentded for important magnetic observations.

3. Irregular Variation : The declination of the needle is subject to variations more or less irregular, which may amount to 1° or more due to unpredictable magnetic disturbances or magnetic storms and may occur at any time. These may last for a few hours or even for several days, but they have a tendency to repeat themselves about every 27 days, this being the time of rotation of the sun with reference to the earth. Such violent and abnormal fluctuations of the needle are thought to be caused by electrical radiations from sun spots, from which a stream of protons and electrons are issued causing magnetic disturbances due to the lonisation and electric charge distribution in the upper atmosphere.

RELATION BETWEEN TRUE AND MAGNETIC BEARING : True azimuths differ from magnetic azimuths by the magnitude of the magnetic declination at the time. If the declination is east of north, then all the magnetic azimuths will be less than corresponding true azimuths by the amount of declination; if the declination is west of north, then magnetic azimuth will be larger than corresponding true azimuths by the amount of declination. Conversion between true and magnetic directions is best made by means of azimuths.

Example 11. The magnetic declination on a certain date at a given place is 15°15'W. The magnetic azimuth of a line is 124°20'. What is the true azimuth of the line ?

Solution : The magnetic azimuth is larger than the true azimuth by 15°15', and the true azimuth of the line is therefore 124°20' - 15°15' = **109°05'.**

Example 12. The magnetic bearing of a line was observed as **S 12°30' E** when the magnetic declination was 22° E. What is the true bearing of the line ?

Solution : This problem is solved by making the conversion by azimuths. The work follows :

Magnetic bearing	=	S 12°30' E
Magnetic azimuth	=	167°30'
Delination	=	+ 22°
True azimuth	=	189°30'
True bearing	=	**S 9°30' W**

Example 13. The magnetic bearing of a line was recorded as S 80°15' W in 1920 at a place which had declination of 14° E in that year. What is the present magnetic bearing if the declination is now 2°30' E ?

Solution : The reduction to true azimuth in this problem is made in the following manner -

Magnetic bearing in 1920	=	S 80°15' W
Magnetic azimuth in 1920	=	260°15'
Declination in 1920	=	+14°
True azimuth	=	274°15'
Declination at present	=	– 2°30'
Magnetic azimuth at present	=	271°45'
Magnetic bearing at present	=	**N 88°15' W**

Example 14. The true azimuth, from north, of a line is given as 144°15'. This line is to be traced by using a compass when the declination is 3°30'W. What should be the reading on the compass to observe this line ?

Solution : The computations follow -

True Azimuth	=	144°15'
Declination	=	+ 3°30'
Magnetic azimuth	=	147°45'
Magnetic bearing	=	**S 32°15' E**

GRID NORTH LINES : There are lines which are parallel to the plotting meridian in a system where the surveys are plotted by rectangular co-ordinates. Any one such line may be a true north meridian through the points along it, but all the other parallel grid lines will differ from the local geographical meridians or true north lines because the latter converge towards the north,

whereas the **grid lines are strictly parallel.**

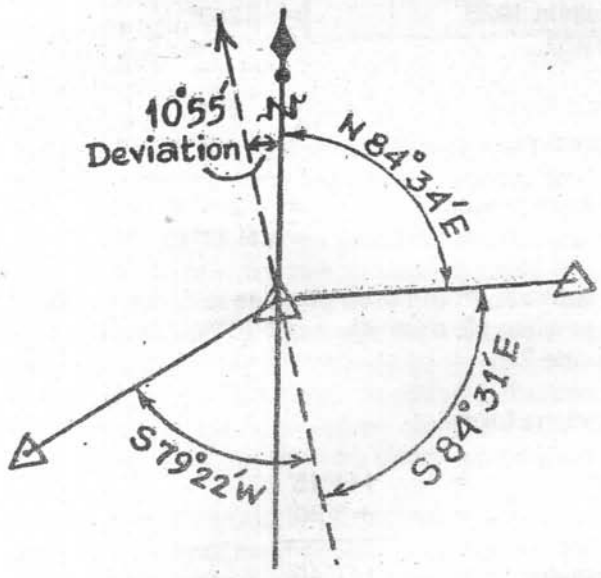
When the mine plans are co-ordinated with those of the Ordnance Surveys, they are plotted, not with reference to the local geographical meridian at the colliery, but with reference to the lines parallel to grid north lines. Such lines are laid down from data relating to O.S. trigonometrical stations in the area and the local geographical meridian at the mine is not required to be known. In mine surveying, grid north lines may be used exactly as though they were true north lines, and they have all the advantages of true north lines in that they are fixed and never alter. **A system of surveying based on the National Grid has the advantages** that all the mine surveys in given area are automatically related to each other and to the ordnance survey, instead of being separate, unrelated surveys, each based on its own, purely local, geographical meridian.

Example 15. The magnetic bearings of two lines AB and AD are respectively S 84°31' E and S 79°22' W. The Ordnance Grid Bearing of AB is N 84°34' E. Find out the Ordnance Grid Bearing of AD.

Solution : The magnetic azimuth of the line AB = 180° - 84°31' = 95°29'.

The ordnance grid azimuth of AB = 84°34'.

The deviation between the two grids = 95°29' - 84°34' = 10°55'. The magnetic bearing of the line being more, hence the magnetic north is 10°55' West of Ordanance grid.



The magnetic azimuth of AD = 180° + 79°22' = 259°22'

The ordnance azimuth of AD = 259°22' - 10°55' = 248°27'.

∴ The Ordnance Grid Bearing of AD = 248°27' - 180° = **S 68° 27' W.**

Example 16. It is desired to correlate an underground traverse with the Ordnance Survey

Grid North of the area. With that object, magnetic observations have been taken simultaneously underground and on the surface, and the records are as follows -

Number of Line	Azimuth Bearing of Underground Line from an Assumed Base	Magnetic Bearing of Line	Magnetic Declination (from Grid North) taken on surface
1	10° 11'	51° 33'	13° 57' West
4	247° 14'	288° 31'	13° 54' West
7	183° 19'	224° 22'	13° 54' West
10	90° 31'	131° 46'	13° 54' West
15	50° 20'	91° 36'	13° 55' West

Calculate the azimuth of the underground base relative to Grid North from the above data.

Solution : Considering **Line 1,** the grid azimuth of the assumed base is obtained by diminishing the magnetic bearing of the first line (= 51° 33') by the sum of the magnetic declinantion (= 13° 57') and the angle from the assumed base (10° 11'). A similar declination applies to all other lines, and the results may be set out as follows -

Grid Azimuth of Base = Magnetic Bearing of line - (Mag. Decl + Assumed Azimuth)

For line 1, Azimuth of Base = 51° 33' - (13° 57' + 10° 11') = 27° 25'

For line 4, Azimuth of Base = 288° 31' - (13° 54' + 247° 14') = 27° 23'

For line 7, Azimuth of Base = 224° 22' - (13° 54' + 183° 19') = 27° 09'

For line 10, Azimuth of Base = 131° 46' - (13° 54' + 90° 31') = 27° 21'

For line 15, Azimuth of Base = 91° 36' - (13° 55' + 50° 20') = 27° 21'

From the results, it appears that the line 7 have been affected by local attraction, disturbances of poor manipulation of the instrument. Therefore, the figure 27° 09' should be discarded and the mean of the other four results accepted as correct.

∴ **Azimuth of underground base relative to Grid North**

= (27° 25' + 27° 23' + 27° 21' + 27° 21') ÷ 4 = 27° 22'30"

LOCAL ATTRACTION

A freely suspended mangetic needle points towards magnetic north and south. But sometimes, in addition to the continually changing forces of the earth's magnetic field, purely local forces may come into play and deflect the magnetic needle out of the line of the magnetic meridian. The deflection of the needle from its original position due to the presence of some magnetic substances is known as *local attraction*. Some of the sources of *local attraction* are -

(i) Masses of magnetic rocks or iron ore.
(ii) An electrical field provided by batteries, cables and other equipments.
(iii) Presence of iron and steel structures near to the compass.
(iv) Presence of rails, pipes, drills, etc. used underground.
(v) Presence of iron made instruments and accessories such as chain, arrows, steel tapes, ranging rods, axes, iron hammer etc.
(vi) Presence of knife, watch, steel buckles, bunch of keys, iron buttons, steel bowed spactacles.

All these sources of local attraction from the vicinity of the compass, whether on the ground or on the person should be kept away when observing bearing of lines by a loose-needle.

DETECTION OF LOCAL ATTRACTION : To detct local attraction at a particular place, fore and back bearings of each line are taken. If the difference of two bearings come out to be 180°, then there is no local attraction at either station. On the other hand if the difference is other than 180°, the observation may be repeated to find out that the discrepancy may not be due to the presence of local attraction in the vicinity. But if the difference still remains the local attraction exists at one or at both the stations.

Example 17. A new roadway is to be set out on a bearing of S 35° W from a point A in the existing roadway AB running N 89° E from A to B. The point A is surrounded with material that attract the needle and the needle indicates the bearing of AB as S 89°E. How would you set out the new road if you had only a loose-needle dial ?

(Survey Comp. 1981)

Solution : Let AC is the new roadway to be set out from A on a bearing of S 35°W = an azimuth of 215°.

AB is the existing roadway on a bearing N 89°E = azimuth of 89°.

Due to local attraction at A, the needle is displaced and the dial reading for AB is S 89°E = 91° azimuth, which is 2° too much. It thus follows that, when the dial is directed to C, it will still read 2° too much, namely 217° or S 37° W, instead of 215°. **To set out the new road,** therefore, the dial would be set up at A and the sights directed in an apparent direction 217° or S 37° W. The correct bearing of the line would then be 215° or S 35° W as required.

Example 18. The following bearings were taken with loose-needle (graduated anti-clock-

wise) during a survey by Candler's system. Stations A and F were free from local attraction. Calculate the correct quadrant bearings of the various drafts.

Line	Foresight	Backsight
AB	287° 54'	286° 45'
BC	141° 27'	142° 57'
CD	91° 15'	91° 48'
DE	71° 24'	69° 12'
EF	46° 33'	47° 51'

(Survey Comp. 1985)

Solution : The first foresight AB is a correct magnetic azimuth 287° 54' = N 72° 06' W. The backsight of AB namely 286° 45', is 1° 09' too small (287° 54' – 286° 45') and 1° 09' must therefore be added to the next foresight BC. Hence corrected foresight BC = 141° 27'+ 1° 09' = 142° 36' = **S 37° 24'E.**

The backsight of BC, 142° 57', is 0° 21' greater and the next foresight CD must be reduced by this amount. The corrected goresight CD is thus 91° 15' – 0° 21' = 90° 54' = **S 89° 06' E.**

The backsight of CD, 91° 48', is 0° 54' greater and hence the next foresight DE = 71° 24' – 0° 54' = 70° 30' = **N 70° 30' E.**

The backsight of DE, 69° 12', is 1° 18' too small, and so the next foresight EF must be increased to 46° 33' + 1° 18' = 47° 51' = **N 47° 51' E.**

The calculations have been correctly performed is shown by the fact that the final calculated azimuth of EF is the same as the final observed backsight of F, this station being specified as free from local attraction.

The results may be tabulated as follows -

(1) Line	(2) Foresight	(3) Backsight	(4) Correction	(5) Azimuth	(6) Quadrant Bearing	(7) Remarks
AB	287° 54'	286° 45'	+ 1° 09'	287° 54'	N 72° 06' W	Stn. A free from attraction.
BC	141° 27'	142° 57'	– 0° 21'	142° 36'	S 37° 24' E	
CD	91° 15'	91° 48'	– 0° 54'	90° 54'	S 89° 06' E	
DE	71° 24'	69° 12'	+ 1° 18'	70° 30'	N 70° 30' E	
EF	46° 33'	47° 51'		47° 51'	N 47° 51' E	Stn. F. free from attraction.

Example 19. The following bearings were taken with a loose needle (graduation anti-clockwise) during a survey by candler's system, stations A and F are free from local attraction. Calculate correct quadrant bearing of each line.

Line	F. S.	B. S
AB	37° 40'	37° 20'
BC	334° 20'	334° 55'
CD	85° 10'	85° 20'
DE	140° 50'	140° 35'
EF	190° 20'	190° 10'

(Survey Comp. 1963)

Solution : The calculations are represented in the table below -

Line	Foresight	Backsight	Correction	Corrected Bearing	Quadrant Bearing	Remarks
AB	37° 40'	37° 20'	+ 20'	37° 40'	N 37° 40' E	Stn. A free from attraction.
BC	334° 20'	334° 55'	– 15'	334° 40'	N 25° 20' W	
CD	85° 10'	85° 20'	– 25'	84° 55'	N 84° 55' E	
DE	140° 50'	140° 35'	– 10'	140° 25'	S 39° 35' E	
EF	190° 20'	190° 10'	–	190° 10'	S 10° 10' W	Stn. F. free from attraction.

Example 20. A close compass traverse ABCD was conducted round a lake and the following bearings were obtained. Determine which of the stations are suffering from local attraction and give the values of the corrected bearings.

Line	Fore bearing	Back bearing
AB	74° 20'	256° 00'
BC	107° 20'	286° 20'
CD	224° 50'	44° 50'
DE	306° 40'	126° 00'

(A.M.I.E. 1976)

Solution : It will be seen from the available data that the difference of fore bearing and back bearing of the line CD is 180°. Therefore stations C and D are free from local attraction. The stations A and B are affected from local attraction as the difference of fore bearing and back bearing of AB and BC is not equal to 180°

Now starting from the line CD, the bearings taken from stations, C and D are correct.
Fore bearing of line DA = 306° 40' correct (given).

Correct B. B of line DA = 306° 40' – 180° = 126° 40'
But observed B. B of line DA = 126° 00'
∴ Error at station A = 126° 40' – 126° = 40' (Negative)

Because the observed Back Bearing is less than the corrected one, therefore the error will be negative and correction will be positive.

Observed F. B. of line AB = 74° 20'
Corrected F. B. of line AB = 74° 20' + 40' = 75° 00'
Corrected B. B. of line AB = 75° + 180° = 255° 00'
But observed B. B of line AB = 256° 00'
∴ Error at station B = 256° – 255° = 1° (+ Ve)

Because the observed Back Bearing is more than the corrected one of the error will be positive and the correction will be negative.

Observed F. B. of line BC = 107° 20'
Corrected F. B. of line BC = 107° 20' – 1° = 106° 20'
Corrected B. B. of line BC = 106° 20' + 180° = 286° 20'

Check : The corrected B. B of line BC tallies with the observed B. B. line BC.
The result is represented in the table below -

Line	Observed Bearing		Correction	Corrected Bearing		Remarks
	F. B.	B. B		F. B.	B. B.	
AB	74° 20'	256° 00'	+ 40' at A	75° 00'	255° 00'	Stations A and B are
BC	107° 20'	286° 20'	– 1° at B	106° 20'	286° 20'	affected by
CD	224° 50'	44° 50'	Nil at C	224° 50'	44° 50'	local
DE	306° 40'	126° 00'	Nil at D	306° 40'	126° 40'	attraction.

Example 21. The following are the bearings observed of the lines of a traverse ABCDE. Determine which of the stations are affected by local attraction and the correct bearing of the lines :

Line	Fore bearing	Back bearing
AB	191° 45'	13° 00'
BC	39° 30'	222° 30'
CD	22° 15'	200° 30'
DE	242° 45'	62° 45'
EA	330° 15'	147° 45'

(First Class, Metal, 1981)

Solution : On examining the observed bearings of the line, it will be noticed that difference between back and fore bearings of the line DE is exactly 180°. Hence both stations D and E are free from local attraction and all the bearings measured at these stations are also correct. Thus the observed bearing of line EA = 330° 15' is correct. Correct B. B. of line EA will be 330° 15' – 180° = 150° 15'. Since the observed B.B. of line EA is 147° 45', a correction of (150° 15' – 147° 45') = + 2° 30' will have to be applied to the bearing taken from A. Thus the correct bearing of line AB becomes 191° 45' + 2° 30' = 194° 15'. The correct B. B of AB will, therefore, be 194° 15 - 180° = 14° 15'. But the observed B. B. of AB is 13°, a correction of + 1° 15' will have to be applied to the bearing BC measured at B. Thus the correct bearing of BC becomes 39° 30' + 1° 15' = 40° 45'. The correct B. B. of BC will be 40° 45' + 180° = 220° 45'. Since the observed Back Bearing of BC is 222° 30', a correction of – 1° 45' will have to be applied to the bearing of CD measured at C. The correct bearing of CD will thus be 22° 15' – 1° 45' = 20° 30'. The corrected B. B. of CD will be 20° 30' + 180° = 200° 30', which is the same as observed one, since the station D is not affected by local attraction.

The results may be tabulated as given below -

Line	Observed Bearing		Correction	Corrected Bearing		Remarks
	F. B.	B. B		F. B.	B. B.	
AB	191° 45'	13° 00'	+ 2° 30' at A	194° 15'	14° 15'	Stations A, B and C are affected by local attraction.
BC	39° 30'	222° 30'	+ 1° 15' at B	40° 45'	220° 45'	
CD	22° 15'	200° 30'	– 1° 45' at C	20° 30'	200° 30'	
DE	242° 45'	62° 45'	Nil at D	242° 45'	62° 45'	
EA	330° 15'	147° 45'	Nil at E	330° 15'	150° 15'	

Example 22. Below are bearings observed in clockwise traverse in a quadrantal system :

Line	Fore bearing	Back bearing
AB	S 55° 30' E	N 55° 30' W
BC	N 68° 15' E	S 68° 00' W
CD	N 49° 30' W	S 47° 30' E
DA	S 20° 15' W	N 18° 30' E

At what stations do you suspect local attraction ? Find the correct bearings of the lines and also determine the included angles.

Solution : The quadrantal bearings of the lines are first converted to whole circle bearings and then corrections are applied. Lastly the corrected bearings are converted into quadrantal bearings.

Line	Observed Bearing		Correction	Corrected Bearing		Remarks
	F. B.	B. B		F. B.	B. B.	
AB	124° 30'	304° 30'	Nil at B	124° 30' S55° 30' E	304° 30' N55° 30' W	Stations C and D are affected by local attraction
BC	68° 15'	248° 00'	+ 15' at C	68° 15' N68° 15' E	248° 15' S68° 15' W	
CD	310° 30'	132° 30'	– 1° 45' at D	310° 45' N49° 15' W	130° 45' S49° 15' E	
DA	200° 15'	18° 30'	Nil at A	198° 30' S18° 30' W	18° 30' N18° 30' E	

A rough sketch of the traverse may by drawn to facilitate calculations. We have -

$\angle A = 180^\circ - (55^\circ 30' + 18^\circ 30') = 106^\circ 00'$
$\angle B = 55^\circ 30' + 68^\circ 15' = 123^\circ 45'$
$\angle C = 180^\circ - (68^\circ 15' + 49^\circ 15') = 62^\circ 30'$
$\angle D = 49^\circ 15' + 13^\circ 30' = 67^\circ 45'$

Check : Some of all the angles $\angle A + \angle B + \angle C + \angle D = 360^\circ$ Which is equal to $(2n - 4) = 2 \times 4 - 4 = 4$ right angles $= 4 \times 90^\circ = 360^\circ$ (correct).

Example 23. The following bearings were observed in traversing with a dial in place where local attraction was suspected :

Line	F. B.	B. B.	Line	F. B.	B. B.
AB	N 45° 45' E	S 46° 10' W	CD	N 29° 45' E	S 29° 10 W
BC	S 83° 05' E	N 82° 55' W	DE	N 35° 12' W	S 35° 12' E

Mention which stations were affected by local attraction and determine the corrected bearings.

Solution : If the calculations are in quadrantal system, positive corrections are applied clockwise and negative corrections counter - clockwise.

By inspection of the observed bearings, stations D and E are found free from local

attraction and hence bearings of DE, ED and DC are correct. The correct bearing of CD will therefore, be N 29° 10' E. Since the observed bearing of CD is N 29° 45' E, the magnetic needle at C is deflected by 35' towards West form North, The corrected bearing of CB (observed at station C) will, therefore, be N 82° 55' W + 35' = N 83° 30' W.

The corrected bearing of BC will be S 83° 30' E. But the observed bearing of BC is S 83° 05' E, the needle is, therefore, deflected by 25' towards East from South. Hence the corrected bearing of BA will be S 46° 10' W – 25' = S 45° 45' W. The bearing of line AB will be N 45° 45' E, which is the same as the bearing of AB ovserved at station A, station A is, therefore, not affected by local attraction.

Example 24. The following bearings were observed in running a closed traverse :

Line	Fore bearing	Back bearing
AB	75° 30'	254° 45'
BC	110° 30'	291° 15'
CD	169° 30'	350° 15'
DE	224° 15'	44° 15'
EA	305° 15'	124° 30'

Indicate which stations are affected by local attraction. Determine the correct magnetic bearings. Also find the true bearings if the magnetic declination was 3° 30' E.

Solution : By inspection of the observed bearings, it is found that the difference between F. B. and B. B. of DE is exactly 180°. Stations D and E are, therefore, free from local attraction. The magnetic bearings taken from D and E are correct. Thus the observed bearing EA = 305° 15' is correct.

Correct	F. B. of line	EA	= 305° 15'
Correct	B. B of line	EA	= 305° 15' – 180° = 125° 15'
Observed	B. B. of line	EA	= 124° 30'
	Correction	= 125° 15' – 124° 30'	= **+ 45' at A**
Observed	F. B. of	AB	= 75° 30'
Correct	F. B. of	AB	= 75° 30' + 45' = 76° 15'
Correct	B. B. of	AB	= 76° 15' + 180° = 256° 15'
Observed	B. B. of	AB	= 254° 45'
	Correction	= 256° 15' – 254° 45'	= **+ 1° 30' At B**
Observed	F. B. of	BC	= 110° 30'
Correct	F. B. of	BC	= 110° 30' + 1° 30' = 112° 00'
Correct	B. B. of	BC	= 112° 00' + 180° = 292° 00'

Observed B. B. of BC = 291° 15'
Correction = 292° - 291° 115' = + 45' At C
Observed F. B. of CD = 169° 30'
Correct F. B. of CD = 169° 30' + 45' = 170° 15'
Correct B. B. of CD = 170° 15' + 180° = 350° 15'

Check : The calculated B. B. of line CD is the same as the observed bearing.

Since the magnetic declination is 3° 30' towards East, the true bearings of all the lines will be determined by adding 3° 30' to the corrected bearing of the lines.

The results may be tabulated as follows -

Line	Observed		Correcteon	Corrected		Dec ination	True		Remarks
	F. B.	B. B.		F. B.	B. B.		F. B.	B. B.	
AB	75° 30'	254° 45'	+ 45' at A	76° 15'	256° 15'	3° 30' E	79° 45'	259° 45'	Stn. D
BC	110° 30'	291° 15'	+ 30' at B	112° 00'	292° 00'	to be	115° 30'	295° 30'	and E are
CD	169° 30'	350° 15'	+ 45' at C	170° 15'	350° 15'	added	173° 45'	353° 45'	free from
DE	224° 15'	44° 15'	Nil at D	224° 15'	44° 15'		227° 45'	47° 45'	local
EA	305° 15'	124° 30'	Nil at E	305° 15'	125° 15'		308° 45'	128° 45'	attraction

LAYING DOWN MAGNETIC MERIDIAN ON OLD PLAN

If any old plan does not show true or magnetic meridian, or it shows a magnetic meridian marked upon it without any date, which is useless, the question of laying down the magnetic meridian may arise.

In such cases, a long well-marked base-line, called meridian test-line, between two accessible prominent objects clearly shown on the plan is to be established on the suface. The magnetic bearing of this baseline is then carefully observed at intervals of 10 minutes over an exact hour with a tested vernier dial or a theodolite fitted with tublular compass. The mean of the bearings observed is accepted as the correct bearing. A new surface survey of the main features shown on the plan including the base line is also carried out.

The new survey is then plotted on a tracing paper to the same scale as that of the old plan allowing for shrinkage, if any. The magnetic meridian is drawn lightly across the plotting. The newly plotted tracing paper is superimposed on the old plan so that surface features coincide. The magnetic meridian is pricked through and transferred to the old plan. The magnetic meridian is then dated.

MAGNETIC INSTRUMENTS

The instruments incorporating the magentic needle include the simple compass, the prismatic compass, tubular compass and surveyor's compass or dial.

PRISMATIC COMPASS : The prismatic compass is a small portable surveying instrument capable of measuring the magnetic bearings of survey lines. The accuracy of the instrument is limited and consequently its application. It is used for surface work, particularly in triangulation reconnaissance.

The instrument consists of a circular box with a pivoted magnetic needle inside. Fixed to the needle is a circular graduated aluminium ring with the zero of the graduations on the south pole of the magnetic needle. The figures are inscribed on the card in reverse so that when viewed through a mirror they can be read correctly. The leading sight consists of a hinged plate with a cross-hair in an aperture or a hair-line scribed on a grass plate. The rear sight consists of a slit and a prism. The observer lines up the sights with the survey stations and then looks through the prism to observe the magnetic azimuth of the line.

The instrument can either be held in the hand or a tripod can be used. The instrument must be approximately level when reading. No provision is made for accucately levelling the instrument.

TROUGH COMPASS : This is so called because the magnetic needle is in a rectangular box or trough. It admits the use of a longer needle mounted on a pivot in the box, the movement being restricted on either side of an index mark. A magnifying glass may be used to see when the needle exactly coincides with the index mark.

TUBULAR COMPASS : This is the most precise type of compass as yet designed for surveying instruments. It consists of a cylindrical brass tube which is totally enclosed to make it dust-proof. The needle about 8 cms long is of thin rectangular section except at the ends where it is triangular in section, the fine edge being uppermost. A bracket, attached to the tube, allows for fitting it to a theodolite.

A glass diaphragm, ruled with two fine vertical lines is fitted to the tube close to the south end of the needle, and the same end of the tube is fitted with an eye-piece, with a magnification of 20, which enables an observer to centralise the end of the needle accurately between the two vertical lines, to within one minute of arc.

A small prism mounted in the top of the tube transmits the illumination for the diaphragm lines. The needle may be raised by a lifter controlled by a milled head screw which is fitted at the other end of the tube.

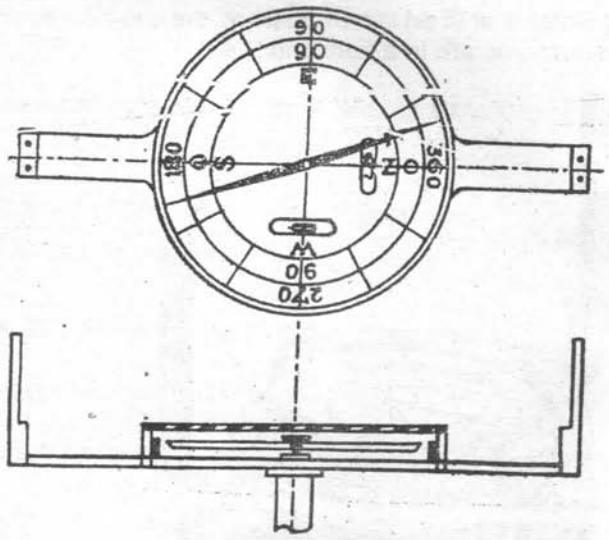
MINER'S DIAL : The miner's dial has been a popular instrument for underground surveying for a considerable time. The use of the dial is now being superseded by the optical mining transits but even so the dial still finds a considerable use. The dial is not nearly as accurate as the theodolite : three minute to one minute dials are the most usual. The design of the dial is really a modified more accurate compass although the use of the magnetic needle has declined over the years because of the increased presence of disturbing influences to be found in the mine roadways.

In its simplest form, the miner's dial consists of a shallow, circular brass box, 13 to 15 cms in diameter and graduated in degrees from 0^{0} to 360^{0}. Within the outer graduated ring is a second brass ring divided into quadrants and degrees, each quadrant being divided 0^{0} to

90° from north and south to east and west. The north and south line of the compass coincides with the line of sight of the dial. It will be noticed that the east and west points of the dial are on the opposite sides of the north-south line to which they occupy on the ground or on a map.

A fine steel point is fixed to the centre of the graduated circles on which balances in a horizontal position a **magnetic needle** (compensated for dip). The needle usually has pointed ends to facilitate reading off on the graduated circles.

The circular brass box has **glass cover** and sometimes a brass lid which protects the instrument for dirt and damage. The instrument is usually provided with a **needle lifter** or clamp by which the magnetic needle may be lifted off the fine steel point and held up against the underside of the glass cover when not in use.



Simple Loose-needle Dial
(graduated anti-clockwise)

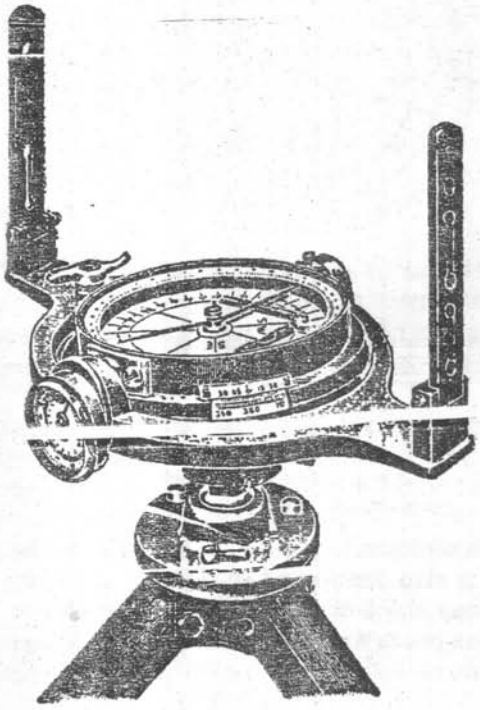
Two spirit levels are attached to the bottom plate of the instument, one on the north-south line and the other on the east-west line. The dial is also fitted with two sighting vanes or sights, which are co-linear with the north-south line. They stand vertical when the instrument is in use but may be folded down over the glass or brass cover for ease of handling when not in use.

The lower half of one sight has a narrow slit cut in it and the upper half has a fine wire or horsehair bisecting a wider opening or window. The other sight has similar slits but transposed in position so that observations may be taken through the sighting vanes in both directions.

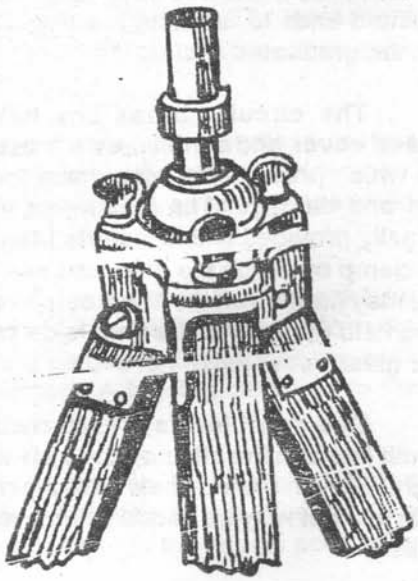
The dial is attached to a tripod and, by means of a ball-and-socket joint and in conjunction with the two spirit levels, the instrument can be fixed horizontally. The instrument may be rotated about its vertical axis, a clamp being provided to fix the line of sight of the instrument in any required direction. On more elaborate dials there is also a "slow motion" or tangent screw to enable the sights to be more accurately fixed on the mark being sighted.

On the more elaborte dials the arms, on which the sights are fixed, are carried on a brass ring (gimbal ring) encircling the compass box and resting on two horizontal trunnions or axis placed along the east-west line. The sights can be locked in a horizontal postition by a locking clip or can be turned through a vertical plane on their horizontal axis, always maintaining the same vertical plane as the north south line of the dial. It is thus possible to take

sights along steeper roadways. A graduated vertical axis mounted on the dial in conjunction with a pointer which moves with the sighting vanes, enables the inclination of the line of sight to be measured in degrees. It is in this connection that the round holes in the sighting vanes are used. Each pair of sights consists of a very small round eye-hole in one vain and a larger hole with horizontal vertical cross hair in the other vane. When the dial face is levelled and the pointer is at 0° on the vertical arc, the eye-hole in one vane and the horizontal cross hair in the other vane are in a horizontal line.



Hedley Dial



Ball-and-socket joint

Another feature of the improved dials (Hedley Dial) is the fitting of a varnier by which angular measurements may be determined more accurately than by the use of a graduated circle alone. By means of a vernier, it is possible to measure an angle by a dial to one minute of arc. The vernier is usually attached to the outside of the compass box.

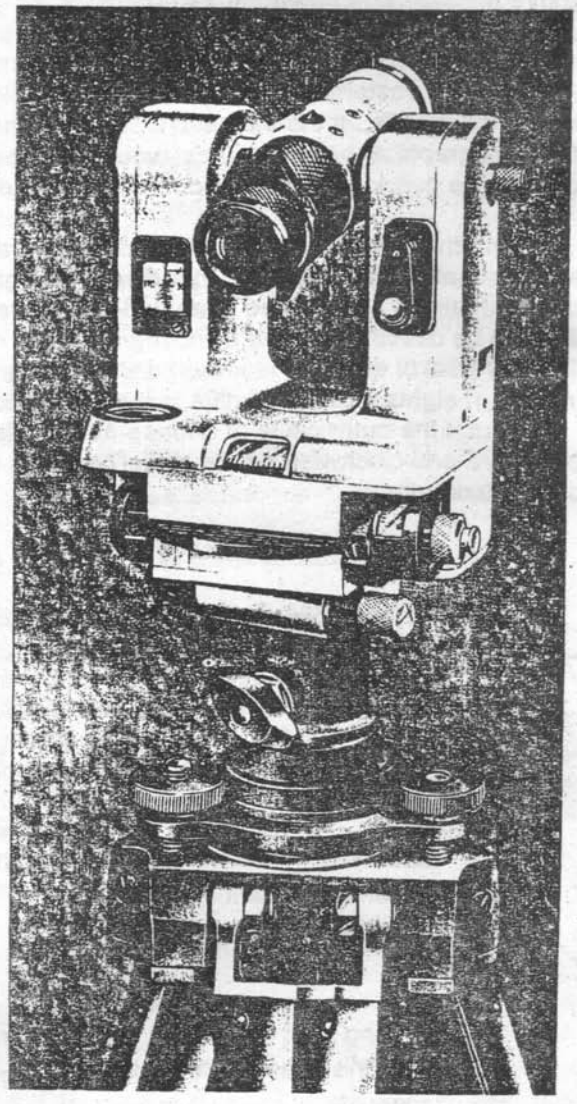
Further improvements to lthe miner's dial are the sighting vanes have been replaced by a telescope supported on an arm fastend to the side of the compass box. The vertical circle (to the right to the telescope) is read by a verneir through a magnifying glass and the vertical circle is fitted with a clamp and tangent screw.

The horizontal circle is graduated to read to one minute of arc by the vernier, which is sighted through a magnifying eye-piece. The reading edge of the circle and the verniers are set at an angle to facilitate reading.

The instrument is set on a ball and socket levelling head fitted with adjusting foot screws.

Optical Sighting Dial : A modern mining dial is shown in figure. The principles of the instrument are basically the same as those of the simple mining dial except the quadrant bearings cannot be read directly from th instrument, but only azimuths.

The upper part of the instrument (telescope, vertical circle, etc.) revolves about a scale graduated to two degrees with a vernier readings to two minutes. The circle graduation is seen through a window in the cover plate. The vertical axis is provided with a clamp and tangent or slow motion screw.



Optical transit dial

The vertical circle is graduated in degrees and read by vernier to five minutes. A friction clamp is fitted to the transit or horizontal axis. The graduations of the vertical circle are seen through the window to the left of the telescope. A circular spirit level provides the index for the taking of inclined angles.

The telescope gives an inverted image with magnification 4x and focussing down to 1.2m. External sights for approximate alignment with the distance station are fitted to the telescope which also has a centering thorn. The telescope can be rotated through 360 degrees through the horizontal axis.

The magnetic needle is housed within the right-hand telescope standard. When the telescope is set on the magnetic meridian the images of each end of the needle appear to coincide and a viewing magnifier is provided. The light for illuminating the needle enters through an aperture on the right hand side of the instrument.

The tripod to which the instrument is attached has a spherical levelling joint and three levelling screws.

A complete miner's dial outfit may include two auxiliary tripods and lamp cups with the usual levelling arrangments. The necessity to remove the dial and tripod from station to station is then avoided as the auxiliary tripods may be centered over or under the back and forward stations and move forward as the survey proceeds.

Why E. and W. Point are Transposed : If a person stands facing magnetic north the east will lie to his right and the west to his left-hand. On a mining dial of the older type, the east and west points occupy the reverse positions and the reason for this described below.

Let us suppose that a miner's dial is set up at this point and the magnetic needle released and allowed to come to rest under the influence of the earth's magnetic field. Next, move the dial round until the zero point of the graduated circle is coincident with the north point of the needle. If the instrument is turned in a clockwise direction through 90 degrees, then, on looking through the sights from the south to north, it is obvious that the person will be looking in an easterly direction and that east point of the dial will be coincident with the north end of the needle,. If the east and west points have been engraved on the face of the dial in their geographical positions, west would have been coincident with the north end of the needle and the bearing read off as due west instead of due east.

In other words, the graduated dial face revolves round a magnetic needle which lies stationary along the magnetic meridian. The sights are attached to the graduated circle, and the graduations are marked in an anti-clockwise direction so that the magnetic bearing of a line can be directly obtained by noting the circle reading opposite the north end of the needle. This reversal of direction is required since the graduated circle moves relative to the needle, instead of sights moving relative to the graduations.

But if the outer circle of a loose-needle dial is graduated clockwise, the bearings indicated are anti-clockwise whole-circle bearings and must be deducted from 360° to obtain correct azimuths.

PRISMATIC COMPASS AND MINER'S DIAL

The difference between a prismatic compass and miner's dial is tabulated below -

Prismatic Compass	Miner's Dial
1. It can be used with hand and without a stand.	1. It cannot be used without a stand.
2. The needle is of broad form.	2. The needle is of edge-bar form.
3. The card and the needle are attached and constant in direction when the prism box and sight vane move.	3. The graduated card being fixed moves with the sights, while the needle not being attached with the card always points to the magnetic north.
4. The sighting of an object and reading of the bearing are done simultaneously.	4. The observer has to sight first and then go round and read the bearing with naked eye.
5. The zero point is at the south point of the needle under the prism when the needle points north.	5. The zero or 360° is at the north of the card in a line with the horse-hair.

Prismatic Compass	Miner's Dial
6. Graduations are written upside down.	6. Graduations are written ordinarily and not upside down.
7. Graduations are from 0^0 to 360^0 clockwise.	7. Graduations are made anti-clockwise. East and West points are transposed since the graduated circle moves relative to the needle.
8. Construction is very simple and only the horizontal circle is provided. The least value that can be estimated is 15 minutes.	8. Some dials are provided with (i) graduated vertical arc to indicate dip and rise in degrees, and (ii) telescope for sighting in preference to the less accurate slit and window sights. Modern dials are microptic type and can read up to 1 minute.
9. It is lighter and used for filling up small details and making rough traverse surveys.	9. It is heavier and is used for more precise works.

Why Zero is Marked at South End of the Graduated Circle in Prismatic Compass ?

When a magnetic needle is freely suspended, it points towards magnetic North and magnetic South. Similarly in case of prismatic compass the graduated circle to which is attached a magnetic needle, will point towards North and South when left freely over the pivot. Also when the object vane points exactly towards the North end of the needle, the beearing of the North direction must be zero. Since the prism is fitted exactly opposite to the object vane and the reading is taken through the prism, therefore, the south end of the graduated circle must be under the prism. Hence the zero graduation of graduated circle must start at South end. As the graduations are marked clockwise, therefore 90^0 must be marked at West end, 180^0 at North end, 270^0 at East end and 360^0 or 0^0 at South end.

The bearings taken in this way will be from the north end in the clockwise direction.

TESTS AND ADJUSTMENTS OF A PRISMATIC COMPASS

Before starting any work the compass should be tested and if not in peamanent adjustment, should be corrected in the following manner :

(i) Needle Adjustment for Balancing : Set and level the compass properly over the station point. See that the magetic needle or compass graduated ring should be horizontal, if not, slide the rider towards the high end so as to make it perfectly horizontal.

(ii) Adjustment of the pivot point : (a) Bring the North end of the needle against the

North of the graduated circle i.e. at 180° . Read the reading of the South end of the needle. If it does not read 0° , correct the error by bending slightly the pivot pin so that the reading on the North and the South end should be 180° and 0°

(b) Now bring the North end of the needle against 90° mark on the graduated circle. Read the reading at the other end of the needle. If it does not read 270° correct the error by bending slightly the pivot pin so that the reading on the North and the South end should be 90° and 270° .

Repeat the points (a) and (b) till perfect.

(iii) Adjustment of Sight Vanes : Set and level the compass properly. Suspend a plumb bob at some distance and sight it through the vanes. If the object vane hair line, eye vane are in the same vertical line, the vanes are vertical, if not, remove the affected vane and either file the higher of the bed or insert a suitable packing under the lower side. Repeat till perfect.

(iv) Needle Adjustment for Sensitivity : To find whether the needle has lost its magnetism or the pivot is blunt, set the compass and sight any object and note the reading. Now disturb the compass and again sight the same object. If the reading is not the same, then the pivot point should be sharpened and the needle should be remagnetised.

(v) Adustment of needle horizontally : Note the readings of both ends of a needle in different positions, if the difference between the readings at both ends is always some constant quantity other than 180°, the needle ends are not in line. The needle is bent horizontally with great care so that the difference between the readings of two ends must be equal 180° for each set of readings.

TESTS AND ADJUSTMENTS OF DIAL

The following tests will be required to see if a dial, whether new or received after overhaul and adjustments, is trustworthy -

1. Magnetic Needle : *The needle should be properly balanced and not sluggish in its movements.* If attracted by a knife blade which is then removed, the needle should settle in the same position as at first.

The needle must be perfectly straight and its opposite ends should differ in their readings by 180° at all points around dial face. If it does not do this, either the needle is not straight or dial is incorrectly graduated or the needle pivot is not central or the agate bearing is damaged. These defects can be rectified only by the makers.

2. Plate Levels : The test is done to find out if the *axis of plate levels is perpendicular in vertical axis.* This may be tested by carefully levelling the dial and noticing if the bubbles remain central after the instrument has been rotated through 180° . If the bubbles remain central all is correct, but if not, they must be brought back to their position half by adjusting screws and half by the foot screws. The bubbles should then remain central in any position of sights.

3. Tests for Sight Vanes : *The line of sight must pass vertically through the instrument centre*. The test may be carried out by folding down the sights and applying a steel straight edge in line with the slits and hairs. The line of sight must also lie in a truly vertical plane when the instrument is set level. To test for the verticality of sights, the dial is levelled over a tripod stand and sighted to a fine suspended plumb-line. The sights are then travelled up and down along the plumb-line. If there is no obliquity of hairs relative to the plumb-line the sights are vertical.

In case of telescopic dials, line of collimation must be perpendicular to the horizontal axis.

4. General : All the clamps, slow motion screws, the needle-lifter and the locking clip should work properly, and the instrument should fit well over the tripod stand. A constant difference of 180° between the two vernier readings should also be noted.

METHOD OF SURVEYING WITH A DIAL

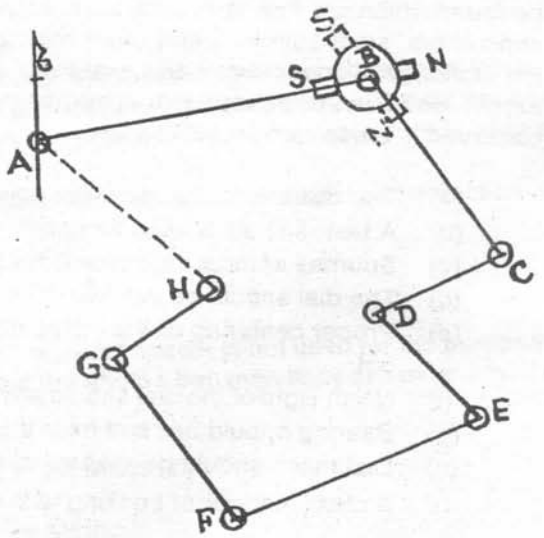
The object of *Dial Surveying* is to cover the area to be surveyed with a number of survey lines and then offsets are taken from them to pick up the detail. The lengths of the lines are measured and sufficient angles taken to fix the positions of survey stations. This system of surveying is called *traversing.* A dial is the most handy instrument commonly used for face surveys, setting out roadways, etc. These can be done in two ways :

(a) Losse-needle method.
(b) Fixed or Fast-needle method.

LOOSE-NEEDLE SURVEYING

This method of surveying may be *(1) Ordinary loose-needle method,* when the area to be surveyed is free from local attraction and the other *(2) Candler's system,* also known as *Reciprocal bearing method or correcting deflection method,* is applied when there are sources of local attraction in the vicinity.

1. Ordinary Loose-Needle Survey : The alternate station method is most common in practice *when there is no local attraction* to affect the needle. The procedure is illustrated in the sketch. The route of the traverse being A to H, via B, C etc. The direction of magnetic meridian is shown by the arrow.



Loose-Needle Survey

The bearing of the line AB is first taken. For this purpose the dial is set up at B and after usual centering (by plumbing directly over or

under the station mark) and levelling (by means of ball-and-socket joint or levelling screws) the magnetic needle is allowed to swing freely until it comes to rest in the magnetic meridian. S-end of the dial is now directed towards the plumb-line at A. The plumb-line can be made clearly visible by holding a piece of tracing paper behind it, and a lamp behind the paper. The N-end is kept in the forward direction. The North end of the needle will now indicate the bearing of AB (not BA). The reading is booked and checked again. Let it is N 80° E.

A plumb line is suspended from the station C. The N-end of the dial is then directed on the forward direction and applying the eye through the S-sight, the sight along BC is aligned. The bearing of the line BC is booked. The distances AB and BC are measured with chain, and offsets are taken by tape.

The dial is then set up at D and the same procedure is adopted. The process is repeated upto station H. A check on the accuracy of work may be done in case of a closed traverse. A brass hook or plug is left on the roof as Dial Mark for subsequent surveys.

Equipment required : A miner's dial with tripod stand; A chain and a tape; Two or three plumb lines; brass hooks; Lamp and tracing paper; Chalk; Paint; Survey book; Pencil, etc.

Arrows and ranging rods will be required in place of lamps if the traverse is made at the surface.

Precautions Required in Orninary Loose-Needle Surveying

The stations where the magnetic bearings are determined must be free from magnetic disturbances. Under present day conditions in the mine it is unlikely that sufficient stations can be found which are free form local attractions, since, tubs, tracks, conveyors, pipes, electrical conductors, steel supports and other mining machineries are now wide-spread throughout the workings. Consequently, this system of surveying is not used belowground to any great extent. However, for reasonable accuracy in the results, the following precautions should be observed in loose-needle surveying :

(a) The dial should be in proper adjustment.
(b) A tested chain should be used.
(c) Sources of local attraction from the vicinity should be removed.
(d) The dial should be set firm.
(e) Proper centering and levelling of the dial over or under the station must be carried out.
(f) North sight of the dial should always be kept in the forward direction of the survey.
(g) Bearing should be read from the N-end of the needle.
(h) Distances should be measured with great care using the chain taut and straight.
(i) A clear manner of booking with necessary details should be maintained.

2. Loose-needle Survey in Presence of Iron

The procedure differs from the ordinary loose-needle surveying in the following respects :

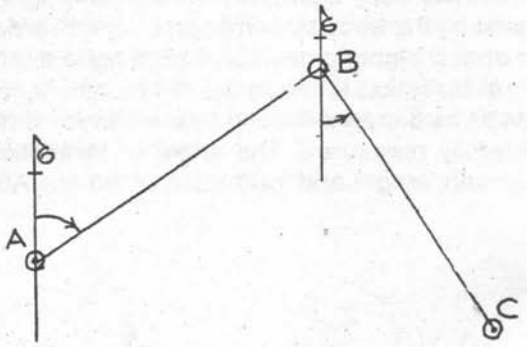
(a) The dial is set up at each station, instead of alternate stations.

(b) Three tripods are usually employed for speed and accuracy.

(c) Each bearing is observed twice, a fore sight from one end of a line and a backsight from the other end of the same line.

(d) There must have at least one station free from local attraction which is known.

The *principle* on which the method depends is that, provided the magnetic attraction at a given station remains the same for both the backsight and foresight from the station, both bearings will be equally false, and if at least one of the stations is free from magnetic attraction, so enabling one correct bearing to be obtained at that station, the correct magnetic bearing of all the remaining lines may be calculated. A loose-needle survey carried out by observing both the fore bearing and back bearing of each traverse line is known as *Candler's System.*



Loose-Needle Survey
(Candler's System)

The dial is set up at station A, centred and levelled,and the magnetic needle is set free. After allowing the needle to settle with the zero of both vernier and compass ring coincident, the dial is revolved until the N-point of the needle also coincides with the zero. The main body clamp is tightened and the vernier released. The sights are rotated so as to observe the plumb-line at the forward station B. The bearing of the line AB is noted after tightening the vernier clamp and making fine bisection of plumb-line with the help of vernier tangent screw.

The vernier is returned to zero and the body clamp released. The dial with its tripod is removed to station B. The bearings BA and BC are obtained in a similar way. The distances of AB and BC are measured and offsets are taken and booked. The same procedure is adopted at other stations in the traverse.

In this method, readings from only one station known to be free from local attraction will suffice to correct for local attraction at other stations.

FIXED OR FAST-NEEDLE SURVEYING

In this method, as the name implies, the magnetic needle is not used for the purpose of determination of bearings of the survey lines but the angles between lines are determined by using verniers. This may be done by -

1. Meridian base-line or Continuous Azimuth method.
2. Double-Foresight Method,
3. Back-and-foresight or Deflection Angle Method.

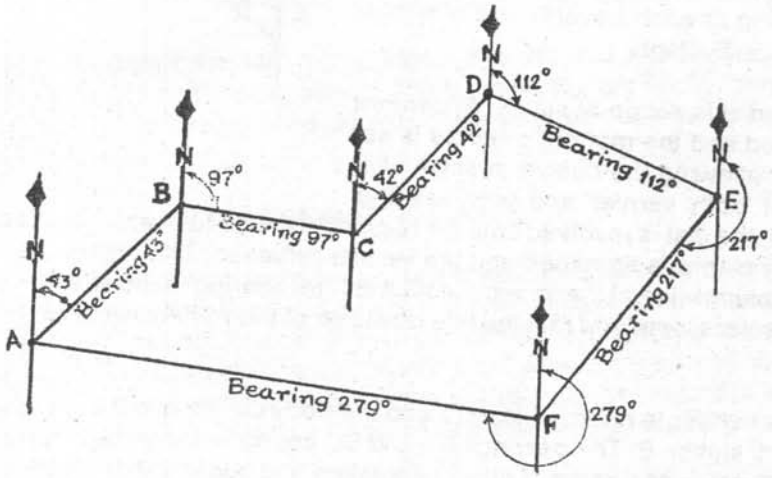
Meridian Base-Line or Continuous Azimuth Method

In this method the survey is completed more quickly if three tripods are used. Three

tripods are desirable because the dial is set at every station, each station in turn being occupied by the forward signal, the dial and the rear signal. *The use of three tripods conduces to speed and accuracy.*

The dial is set up at A, one extremity of the base line of known bearing, and with the vernier set and clamped in that magnetic azimuth, the signal at the other extremity of the line is sighted.

The body clamp is now carefully tightened and a fine bisection of the station signal is made by the body slow-motion tangent screw. The vernier is released and the sighting vanes, or optical sighting device (according to the make of dial), is derected at the signal station B. Final bisection is made by the vernier tangent screw. The azimuth bearing of the line AB (=43°) as denoted directly by the vernier is read. The distance from A to B and the offsets are carefully measured. The angle of inclination of the line is read from the vertical arc. The azimuth, length and inclination of the line AB are recorded in the field-book.



Meridian Base-line Method of Dialing

With the vernier still clamped, the dial is released from the spindle of tripod at A and removed it to station B, where the vernier leading the general direction of the Survey, a backsight is taken to station A, the main body clamp is tightened and the bisection again obtained by the lower tangent screw. From station B towards C the sequence of operations is a repetition of those at A towards B, and so on. While sighting the station C, the tripod from station A is removed to station D.

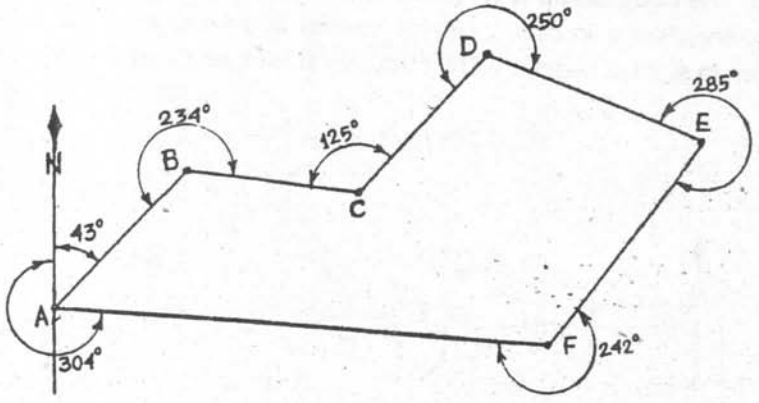
Care must be taken in backsighting to make sure that only the main body clamp and body tangent (or lower tangent) screws are used, whereas in taking foresight the vernier clamp and its tangent are operated. On some dials the heads of the screws are different shape to avoid confusion.

The magnetic meridian base-line method is simple, speedy and reasonably accurate.

Office work is a minimum and gross errors are readily detected. If a single error occurs, however, every succeeding step is vitiated by it and the effect is cumulative as the survey proceeds.

Double-Foresight Method

This, as the name implies, treats both sights as foresights. At station B the dial is set and levelled (the bearing of BA having been previously determined) and the vernier and the compass plates are locked at zero. In this method the eye of the surveyor is, for all observations, away from the inside vernier or next to the outside one. With body clamp loose a sight is taken to station A. The body clamp is tighened and the vernier, now released, is rotated until C is sighted. The whole-circle bearing, as read, gives the angle ABC as indicated in the figure. If the angle is measured several times at the same station, the mean of such a series of repetitions will give a reading of somewhat greater accuracy.



Double-foresight Method
(exterior angles measured in clockwise traverse)

The dial when transferred to station C, is directed to B with vernier against zero. The angle BCD is then measured. All bisections are made as carefully as possible using tangent screws. Signals should be as fine as possible. All distances between stations are carefully measured, and both angles and distances correctly booked.

In case of double-foresight method, if the survey proceeds in a clockwise direction, the angles measured between the lines are exterior and if the general direction of the survey is counter-clockwise, then the interior angles are measured.

Before the survey can be plotted, the azimuth of each line relative to initial base of the survey must be calculated. Sketch is always advisable as a guide.

The double foresight method is most commonly used in mine surveying for the following reasons -

(i) The magnetic attraction of the needle does not affect the bearings of lines as in Loose-needle Method.

(ii) There is no cumulative errors as the angles are measured independently.

(iii) Angles can be checked by repetition and reiteration methods.

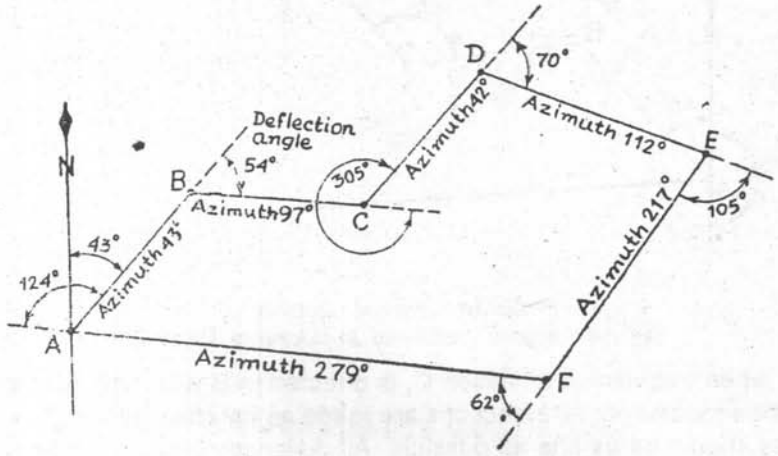
(iv) It is an accurate method, takes less time and three tripods can be used.

(v) The booking of incorrect angle is of remote possibility.

(vi) Office work is minimum. Calculations involve only when bearings of other lines are to be found out.

Back-and-foresight or Deflection Angle Method

The dial is first set at the starting station A and the azimuth of the line AB is determined in the usual way. The dial is then set at station B centred and levelled. The vernier is set to zero and clamped. The body clamp is released and the dial is directed, with the vernier directly under the observer's eye in case of inside vernier and away from the observer in case of outside vernier, to sight at station A. The body is clamped and final bisection is done with body



Back-and-Foresight Method

slow-motion screw. The vernier is released and with vernier away from observer's eye (fore-sight) the station C is sighted. The vernier is clamped and the reading at the vernier which measures the deflection angle (= 54° in the fig.) is found out. The verneir is returned to zero, body clamp released and the dial is removed to station C. Here the movements are repeated. Similar procedure is adopted at other stations. Linear measurements, vertical inclination, etc. are all noted and recorded. The azimuth of any line can be determined by adding deflection angle to the bearing of previous line. If the azimuth obtained by this addition exceeds 360°, from the sum 360° is to be subtracted, which calculation does not affect the direction of the line.

The whole-circle bearings are calculated as follows -

	43°	=	Azimuth of AB
+	54°		
	97°	=	Azimuth of BC
+	305°		
	402°		
−	360°		
	42°	=	Azimuth of CD
+	70°		
	112°	=	Azimuth of DE
+	105°		
	217°	=	Azimuth of EF
+	62°		
	279°	=	Azimuth of FA
+	124°		
	403°		
−	360°		
check =	43°	=	Azimuth of AB

BOOKING THE SURVEY

The booking of underguound survey has always been a matter for individual preference, and so standards regarding any special method have ever been laid down. The survey is booked upwards, starting at the bottom of the page. There are two distinct ways of booking : (a) the notational method which is similar to the double-line type for chain-surveying, and (b) the sketch method. Both have their adherents, advantages and disadvantages. The booking should be accurate, clear and capable of easy interpretation by persons other than those engaged in the actual observations.

A specimen page of Field-Book.

(underground closed traverse at patmohana colliery 5 Dip Section of Barachak Seam with a Microptic Dial and 30m chain.)

	52.33	Stn.2
2.63	51.66	
	43.33	1.56
2.95	28.66	
←	⊗ 26.00	Tie stn (T_1)
1.00	20.60	
	18.00	1.92
1.65	2.33	
8°20'	1.20	1.79
	⊙ Stn. 1	St. 1-2Azimuth = 333°35'

ERRORS IN DIAL SURVEYING

The following are the sources of errors in dial surveying :

1. Insirumental errors.
2. Errors due to Natural Causes.
3. Personal or observational Errors.

1. Instrumental Errors

These errors are due to faulty instrument or due to faulty adjustment. Thus the errors due to these effects are -

(i) The pivot being bent and not in the centre of the graduated circle.
(ii) The needle being sluggish due to loss of magnetism or due to blunt pivot head.
(iii) The needle is not moving freely to its horizontal plane due to the counter weight called rider attached to the needle not properly balanced causing dip.
(iv) The needle is not perfectly straight and the graduated circle not being horizon tal.
(v) The dial is incorrectly graduated.
(vi) The line of sight is not passing through the centre of the graduated circle.
(vii) Vertical hairs are slack and not vertical or these are too thick to bisect properly.
(viii) Clamps and screws are not functioning properly.
(ix) In case of telescopic sight it is out of adjustment or its lenses are defective.

2. Personal or Observational Errors

The errors may arise due to the following reasons during manipulation and handling -

(i) Inaccurate centering and levelling.
(ii) Inaccurate bisection of the object sighted.
(iii) Setting the declination off on the wrong side of the north.
(iv) Declination set off when reading magnetic bearings.
(v) Reading the wrong end of the needle.
(vi) Parallax (reading while looking from the side instead of along it).
(vii) Failing to check the forward and backward bearings when possible.
(viii) Incorrect booking.

3. Errors due to Natural Causes

These errors are due to external influences :

(i) Variation in declination.
(ii) Local attraction.
(iii) Irregular variation due to magnetic storms or earth quakes.

PERMISSIBLE ERRORS IN DIAL SURVEYING

On plotting a closed traverse with respect to field measurements it is seen that the end point of traverse seldom coincides exactly with the starting point. This discrepancy by which the end point does not coincide with the starting points is called closing errors or *errors of closure*.

The closing error may be due to the errors in the field measurements of angles and distances and due to wrong plotting. If the closing error is within some permissible limit, the same is adjusted graphically, but when it exceeds the permissible value the field work has to be repeated.

The limit of error of closure by Miner's dial after plotting shall not exceed 1/1000th of the sum of horizontal lengths of the drafts. The total angular error of a closed traverse should not normally exceed $(20 + x)\ \sqrt{n}$ seconds where 'x' is the least count of the dial and 'n' is the number of stations. The coefficent of $\sqrt{n}$ depends on the method of measuring the angles and on the number of repetitions.

ADVANTAGES AND DISADVANTAGES OF DIAL SURVEY

The following are the advantages and disadvantages of dial survey :

Advantages

1. The magnetic bearings are observed quickly.
2. The dial is light and can be easily handled.
3. The bearing observed from either end of a line.
4. Each bearing is observed independently, the tendency of errors is therefore less.
5. It is useful in overcoming obstacles in chaining.
6. It is suitable for areas which cannot be divided into net work of triangles such as towns, congested areas.

Disadvantages

1. The dial suvey is a rough type of survey and is used where less accuracy is required.
2. It is unsuitable in magnetic areas.
3. Unless care is taken, the bearings observed at places having local attraction will be wrong.

EXERCISES

1. Write short notes on : (a) Maridian (b) Magnetic Storms (c) Magnetic dip (d) Agonic lines (e) Declination. **(Survey Comp 1965)**

2. What do you understand by the term "bearing" of a line ? Convert the following Q.B, into azimuthe bearing : N 15°25' W,, S 27°27' W, N 85°44' E; and the following Azimuth into Q.B : 23°40', 230°45', 169°27', 358°12'. What is relationship between "true bearing" and "magnetic bearing" of a line ?
(**Ans.** 344°35', 207°27', 85°55', and N 23°40'E, S 50°45' W, S 10°33' E, N1°48' W)

3. What is meant by the following : (a) Magnetic Declination, (b) Diurnal variation, (c) Secular variation, and (d) Irregular variation ? Describe an accurate method for determining the magnetic declination. **(Survey Comp. 1957, 1959)**

4. The interior angles of a closed traverse ABCD run anticlockwise are as follows :
$\angle A = 50°10'$, $\angle B = 100°20'$, $\angle C = 120°20'$, and $\angle D = 89°10'$, it the fore bearing of the line BC = N 20°20' E, find out the bearings of all other lines.
(**Ans.** AB = S 80°0' E, CD = N 39°20' W, DA = S 49°50' W)

5. A line was drawn to a magnetic bearing S 57°15' E on an old plan when the magnetic declination at the place was 3° W. To what bearing it should be set now if the magnetic declination is now 6°30' E ? (**Ans.** S 66°45' E))

6. What is the difference between a prismatic compass and a surveyor's compass ? Describe the sources of errors in compass surveying and how the same can be counteracted. **(A.M.I.E. 1977)**

7. What is meant by traverse surveying ? How does it differ form chain surveying ? Distinguish between a closed and an open traverse. **(A.M.I.E. 1975)**

8. (a) Explain why E is placed on the left of N on the dial face. (b) What is the object of graduating a loose-needle dial counter-clockwise ?
Draw a circle assuming it to be face of a loose-needle miner's dial. Show the sights pointing in a direction of S 65° E. **(Survey Comp. 1975, 81)**

9. You have been given a dial which has just been received after overhaul and adjustments. Describe the test you will apply to see the adjustments have been done properly. **(Survey Comp. 1958)**

10. (a) Name and describe briefly the various methods of surveying with dial. (b) Indicate the most suitable method of dial survey in underground workings of a mine. (c) Give a specimen page of the booking for such a survey. **(Survey Comp 1957, 75, 85)**

11. (a) Specify the instruments and other reaquisite needed for an ordianary loose-needle survey. (b) Make a list of the various precautions that should be observed in an ordinary loose-needle survey.

12. Describe one type of vernier dial. Explain with the aid of sketch how you would make a fast-needle survey of a small district by the magnetic meridian base-line method. **(Survey Comp. 1962)**

13. Under what conditions is Continuous Azimuth System of traversing adopted ? Describe with sketch any one of its methods and give reasons for preferring the same to other methods. **(Survey Comp. 1962)**

14. In a closed traverse ABCDEA, the bearing of the line AB is N 35° E. The external angles between succesive lines are as follows :
ABC = 259° 50'; BCD = 225° 40'; CDE = 234° 10'; DEA = 228° 30'; EAB = 251° 50'.
Ascertain (i) Azimuth bearing of each line; and (ii) whether the angular measurements are correct by applying the expression for the sum of the interior angles.

(**Ans.** Azimuths : AB = 35° ; BC = 114° 50'; CD = 160° 30'; CE = 214° 40'; EA = 263°10'. Interior angles = 108° 10' + 100° 10' + 134°20' +123° 50' +131° 30' = 600°. But (2n - 4) x 90° = 540°)

15. The following bearings were observed with a dial :

AB	–	74° 0'	BA	–	254° 0'	DE	177° 0'	ED	–	0° 0
BC	–	91° 0'	CB	–	271° 0'	EA	189° 0'	AE	–	9° 0
CD	–	166° 0'	DC	–	343° 0'					

Where do you suspect the local attraction ? Find the correct bearings. **(A.M.I.E. 1976)**

(**Ans.** Local attraction at D. Corrected bearings DC = 346° 0' and DE = 180° 0')

16. The following fore and back bearings were observed in traversing with a compass in place where local attraction was suspected :

Line	F.B.	B.B	Line	F.B.	B.B.
AB	38° 30'	219° 15'	CD	25° 45'	207° 15'
BC	100° 45'	278° 30'	DE	325° 15'	145° 15'

Find the corrected fore and back bearings and true bearings of each of the lines given that the magnetic declination was 10° W.

(**Ans.**

Line	F.B.	B.B	F.B.
AB	38° 30'	218° 30'	28° 30'
BC	100° 00'	280° 00'	90° 00'
CD	27° 15'	207° 15'	17° 15'
DE	325° 15'	145° 15'	315° 15')

CHAPTER - VII

LEVELLING

★★★

(2) Level Line : A level line is a line lying in a level surface. It is normal to the direction of gravity as indicated by a plumb-line. A level is thus at all points equidistant from the centre of the earth. In effect, it is a circular curve whose radius is its distance from the centre of the earth. In plotting mine sections, parallel straight lines are always used to represent level lines.

(3) Horizontal Surface : The surface through a point which is tangential to the level-surface at that point is known as horizontal surface. It is perpendicular to the plumb-line at that point.

(4) Horizontal Line : A horizontal line through a given point is ordinarily regarded as identical as a level line, but in levelling, it is usual and convenient to regard it as a straight line normal to a plumb-line through the point (but not normal to the direction of gravity throughout its length). It thus forms a trangent to a level line through the same point, although it sensibly coincides with the level line within ordinary limits of sighting. The line of sight through a properly levelled telescope is a horizontal line in the sense defined.

(5) Vertical Surface : A vertical surface through any point is a surface normal (i.e. at right-angles) to the level surface at that point.

(6) Vertical Line : A vertical line through any point is that indicated by a plumb-line freely suspended at the point and, neglecting the irregular distribution of mass in the earth's crust, it points directly to the centre of the earth. Vertical lines are therefore not parallel but radial. For all practical purposes, however they may be regarded as parallel. If two vertical shafts, each 400m deep, 1.6km apart at the surface, the distance between the bottoms of the two shafts will only be about 10cm less than 1.6km. This error is negligible.

(7) Datum Line : A datum line is any horizontal line (representing a horizontal plane) from which the heights or depths of the various staff stations may be measured. It forms a base-line to which the levels of the stations may be referred. The mean sea level is the most convenient datum to assume. For local purposes any convenient plane may be assumed as the datum for the time being. All levels shown on working plan of mines shall be referred to an arbitrary datum, 5000m below the Mean Sea Level as determined by the Survey of India. The levels of underground roadways and workings shall be related to this *National Datum - Mines,* thus ensuring that all underground level figures are positive.

In india, the Ordnance Surveys are reduced to the Great Trigonometrical Survey (G.T.S.) datum at Karachi (value taken as zero) and 8 other stations of undivided India. The height of the G.T.S. Bench Mark at Astronomical Ovservatory, Madras above mean sea level is 6.733 metres.

(8) Mean Sea Level : Mean sea level is the average height of the sea water for all stages of the tides. At a place it is the average of hourly tide heights for a long period of 19 years.

(9) Reduced Level : The height or depths of any points above or below any datum is called the reduced level of that point. Sometimes, it is known as elevation of a point. The difference in elevation is the vertical distance between them.

(10) Bench Marks : The bench marks are fixed points of reference whose elevation above

the datum is known. The bench marks may be of the following types -

(a) Great Trigonometrical Survey Bench Marks : The bench marks, known as G.T.S. Bench Marks, are fixed with high precision at regular intervals throughout the country by the Survey of India Department with reference to mean sea level at Karachi. The position and reduced level of these bench marks are published by the depratment.

(b) Permanent Bench Mark : The Bench mark which are fixed by the Government Agencies such as Public Works Deprtment (P.W.D.) or Military Engineering Services (M.E.S.) are known as permanent bench marks. Such bench marks take the form of a broad arrow pointing upwards to a horizontal line, and are chiselled on some clearly well defined permanent point such as culvert or bridge, kilometre stone, building, or other permanent landmarks. They are of permanent nature. Their value is written and their position is recorded for future reference.

(c) Arbitrary Bench Mark : When levelling operations is carried out the whole work may not be completed in a day. In such cases, at the end of day's work some reference points are fixed, the reduced levels of which are known or calculated. The work is again resumed with reference to such points. These points are known as Temporary bench marks.

(11) Back-Sight : A back-sight in levelling is the first reading taken, after the levelling instrument has been set up, to the initial staff station, or to a staff-station termed the change-point, to which a foresight has just been taken from a previous instrument-station.

(12) Fore-Sight : A fore-sight is the last reading taken to the final staff-station or to a staff-station that is to form a charge-point for a series of readings. It differs from an intermediate sight in that the latter is never taken to change-point.

(13) Intermediate-Sight : The readings taken on all points whose elevations are to be determened and are in between the backsight and foresight, are called intermediate sights.

(14) Change-Point : The point on which both the fore-sight and back-sight are taken is called a change-point. When the levelling instrument is to be shifted from one position to other position in levelling operation such points have to be fixed. The reduced level of such point is to be calculated accurately as these are considered for future reference in the progress of levelling.

(15) Line of Collimation : It is an imaginary straight line which joins the optical cetre of the object-glass with the point of intersection of the cross-hairs of the diaphragm. It coincides with the axis of the telescope and is parallel to the axis of the bubble-tube when the instrument is in correct adjustment. It is also known as line of sight.

(16) Axis of the Bubble-Tube : An imaginary line tangential to longitudinal curve of the bubble tube at its mid-point is known as axis of the bubble-tube. It is always horizontal when the bubble is in the centre of its run.

(17) Height of instrument : The height of the instrument is the elevation of the line of sight with respect to the datum for each set up.

(18) Station : In levelling operation, the station means the staff-station where the levelling staff is held. It is a point whose elevation is to be fixed at a known elevation.

(19) Parallax : The eye-piece of a telescope slides into the end of the inner tube so that it can be adjusted by moving it in or out until the cross-hairs appear sharply defined. If the diaphragm is out of focus with the eye-piece the cross-hairs will be seen indistinctly and if the telescope is focussed on a distant object that object will appear to flit accross the cross-hairs when the observer's eye is moved from side to side of the eye-piece. This apparent movement of the image relative to cross-hairs is known as instrumental prallax. It occurs when the image formed by the object-glass falls short of or beyond the diaphragm. Accurate reading of the levelling staff can not be taken until the parallax has been eliminated. It should be observed that the position of the eye-piece is dependent on the vision of the observer, and consequently before beginning to take staff reading the eye-piece should be properly adjusted.

To eliminate the parallax, the telescope is focussed on a distant object and a piece of white paper then held in front of the object glass. The eye-piece is moved in and out with a screwing motion until the cross-harirs are seen in sharp focus. The telescope is then focussed upon the difined distant object so that the image formed appears clear and sharp. If the adjustment is correct, no relative movement between the image and cross-hairs will exist by moving the eye about looking through the telescope.

METHODS OF LEVELLING

Methods of levelling may be broadly divided into three classes, (1) geometrical methods, (2) trigonometrical method, and (3) the physical method.

Geometrical Method : The geometrical method comprises the direct measurement of the vertical distances between points whose levels are required and a horizontal line or plane set out by means of a spirit level. Normally, it involves the use of a suitably mounted telescope, generally a Dumpy Level, for setting out the horizontal lines, and a graduated staff for measuring the vertical heights; and with these instruments the method is capable of giving the highest degree of accuracy. The Dumpy Level, however, is not suitbale for steep gradients, but, in the absence of an angular instrument, the work may be carried out by means of a wooden Straight-edge, a mason's spirit-level and a tape or staff. The method is also known as *direct levelling.*

Trigonometrical Method : The method utilises an angular instrument having a graduated vertical arc to measure vertical angles of dip or rise, and a chain or steel band to measure distances. Difference of level are then calculated trigonometrically This method, whilst not as accurate as levelling with a dumpy, engineer's or surveyor's level can give good result and, in steep underground roadways, may be the only method available to the surveyor. It is also known as indirect levelling and in a modified form it is called *stadia levelling.*

Physical Method : In this method, differences of level are deduced from the readings of a barometer or of boiling point thermometer. The method is based on the fact that the atmos-

pheric pressure varies inversely as the height of any place. The results, however, are approximate only and the method is of little use in mine surveying.

LEVELLING INSTRUMENTS

The instruments commonly used in direct levelling are (1) a level and (2) a levelling staff.

LEVEL

The purpose of level is to provide a horizontal line of sight. Essentially, it consists of the following four parts :

(a) *A telescope* to provide line of sight.

(b) *A level tube* to make the line of sight horizontal.

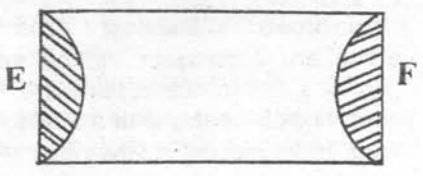
(c) *A levelling head* to bring the bubble in centre of its run.

(d) *A tripod stand* to support instrument

Telescope : The telescope consists of two tubes, one slides into the other and fitted with lens and diaphragm having cross hairs. It creates a line of sight by which the reading on a staff is taken. The essential parts of telescope are - (1) Body, (2) Object glass, (3) Eye piece, (4) Diaphragm, (5) Ray shade, (6) The rack and pinion arrangement, (7) Focussing screw, and (8) Diaphragm screws.

The *objective* or *object glass* of a level (and also of a theodolite) is a *compound lens*. The outer component of the objective is a convex lens of crown glass and the inner a convexo-concave lens of flint glass. This construction has the effect of overcoming two of the defects of a simple lens, known respectively as "Chromatic" and "Spherical" aberration. *Chromatic aberration* is a defect whereby a blurred and coloured image is formed due to rays of white light being split up into their various components and converging to different foci on passing through the lens. *Spherical aberration* is a defect arising from the surface of lens being ground so as to form part of a sphere, with the result that the rays of beam proceeding from a point are not refracted to pass through a single point and a sharp image is not formed. The image formed by the objective is always inverted.

The *eye-piece* is generally of the *Ramsden* type or so-called positive type and consists of two equal plano-covex lenses, called the field lens F, and the eye-lens E respectively placed with their curved surfaces facing each other. Such an eye-piece is very free from chromatic and spherical aberration.



Ramsden Eye-piece

The *diaphragm* consists of a brass ring placed at the comon focus of the object glass and the eye-piece, and carries a "reticule" or "graticule" of cross-hairs or cross-wires. The reticule or cross-hairs

appear magnified when viewed through the eye-piece and therefore must be very fine. These

Various Types of Recticule

are of various forms as shown in figure, and may be spider-web or fine lines ruled on glass, or platinum-iridium point, Spider-webs form very fine and clean line but are delicate and tend to become slack and sag in excessive dampness. Lines on glass retain their position permanently but dew is liable to form on glass with a falling temperature. Platinum-iridium points (as at c) are very durable but they are less convenient for sighting. For levels, the reticule shown at (a) is generally used because it has the advantage as the horizontal hair is used to read the staff and the two vertical hairs enable the surveyor to see if the staff is vertical laterally. Most telescopes are also equipped with two additional horizontal hairs called stadia or subtense line one below and the other above the central horizontal hair and equidistant from it to determine distance by stadia. Reticules shown (a), (b) and (c) are used in levels and forms (d) and (e) are used in theodolites. The diaphragm ring is held in place by two or four capstan screws. The cross-hairs may be attached direct to the ring or secured within an interchangeable cylindrical cells which fits the ring.

A telescope may be of either the external focussing type or the internal-focussing type :

External Focussing Telescope : In the external focussing type, the telescope barrel is formed by two tubes, one sliding with the other . The large outer tube carries the similar eye-piece at its opposite end. This type of telescope has better brilliancy but it has the disadvantage of having a greater length of the telescope.

Internal-Focussing Telescope : In this case, there is only a single tube of fixed length (apart from the adjustable eye-piece) and focussing is performed by altering the position of an internal double-concave lens the effect of which is to render the rays refracted by the object-glass less convergent. The objective and eye-piece are kept fixed with an outer tube and the double-concave lens is mounted in the interior tube which can be moved to and fro between the diaphragm and objective by means of rack and pinion arrangement and an external focussing screw. This type of telescope is used on the more modern levels.

Advantages of internal-focussing telescope over the external-focussing type are as follows :

(1) The overall length of the tube is not altered during focussing. The focussing slide is light in weight and located near and middle of the telescope. Hence the balance of the telescope is not affected and the bubble is not disturbed during focussing.

(2) Wear on the rack and pinion is less.

(3) Line of collimation is less likely to be affected by focussing.

(4) The draw tube is not exposed to weather, the telescope remains free from dust and moisture.

(5) An instrumental constant is also eliminated in making measurements by stadia method.

(6) The negative focussing also gives the optical designer an extra lens to work with in order to reduce the aberrations of the system and to increase the diameter of the objective lens.

Disadvantages of internal-focussing telescope are as follows :

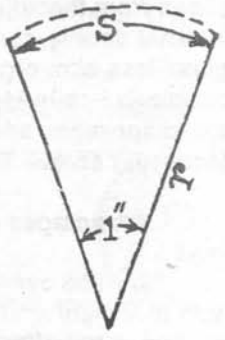
(1) It reduces brilliancy of the image due to the addition of lens.

(2) The interior of the telescope is not easily accessible for field cleaning and repairs.

Levels Tube : The level tube, also known as bubble tube or spirit-level, is a glass tube sealed at both ends, containing a sensitive liquid and small air bubble. The liquid must be non-freezing, quick acting, and relatively stable in length for normal temperature variations. Purified synthetic alcohol has generally replaced the mixture of alcohol and other formerly used. Uniformly spaced graduations, generally 2mm, are etched on the exterior surface of the tube to show the exact position of the bubble. The glass tube is circular in cross-section, and accurately ground internally in lengthways both upper and lower half to form a perfect symmetrical barrel shaped tube. In Dumpy Level the radius of curvature of the tube is generally 18m, but it may be as much as 54m in high class instrument.

The *sensivity of a spirit-level* is its ability to exhibit small deviations from horizontal. It is ascertained by the distance through which the bubble moves for given change of inclination. The more sensitive the level, the greater is the displacement of the bubble for any change of inclination, and more accurate the work. But the pitch of screws, quality of lenses or optical power of the telescope, the graduations of the instruments all rule the sensitiveness of the bubble. The sensitiveness is in proprotion to its length, a long bubble sets much more quickly and accurately than a short one. Small tubes are sluggish and take more time to work. The sensitiveness of a spirit-level is directly proportional to the radius of curvature.

$$\therefore \quad \frac{S}{2\pi r} = \frac{1''}{360^0} ; \qquad \text{Hence} \quad S = \frac{r}{206264.80625}$$



Where, S = length of arc through which the bubble moves for a change of inclination of 1"

r = radius of curvature of the level tube.

Mobility of the liquid with which the tube is filled and the amount of adhesion of the liquid to the glass have also effect on the sensitiveness of the bubble.

The sensitivity of at level is expressed in two ways : by (a) the angle, in seconds, subtended by one division on the scale and (b) the radius of curvature of the tube. If one division subtends an angle of 20" at the centre, it is called a 20" bubble. A 20" bubble having 2mm divisions has a radius of 20m. The sensitivity of level on wye and dumpy levels ranges from 20 to 90", the usual value

being approximately 20" for 2mm divisions. The level bubble used on precise instruments is of a coincident type in which the bubble is centred by bringing the two ends together to form a smooth curve and a prism splits the bubbie to make the two ends visible simultaneously. Because of the opposite motions of the two ends and the magnification provided in an optical-reading instrument, the coincident bubble can be centred with an accuracy of $^1/_{40}$ of its sensitivity. On the other hand, a non-coincident type of bubble used in ordinary Dumpy Level has an accuracy of $^1/_{10}$ of its sensitivity.

Measurement of Sensitiveness of Spirit Level

To find either the radius of curvature of the level tube or the angular value of one division of the level tube :

(i) Select a base line AB, 50 to 100m in length, on a fairly level ground and measure it accurately by steel tape.

(ii) Set up the level at A, one end of the base, and hold the staff at B. the other end.

(iii) With the bubble central, take a staff reading on the staff held vertically at B.

(iv) Move the bubble over 3 to 4 divisions from its centre by turning the levelling screws and again read the staff at B.

(v) Find the difference between two staff readings.

Now, The angle of inclinatton between the lines of sight in radians,

$$\alpha = \frac{S}{D} = \frac{nl}{R}$$; where, S = difference between the two staff readings;

D = the distance from the instrument to the staff; n = the number of divisions through which centre of the bubble is moved; l = length of one division on the bubble tube (normally 2mm); and R = the radius of curvature of the bubble tube.

The angular value of one division in radians = $\frac{l}{R} = \frac{S}{Dn}$ radians

or, $\frac{1 \times 206265}{R} = \frac{S \times 206265}{Dn}$ seconds

(1 radian = 206,265 seconds)

Example 1. A bubble tube has sensitiveness of 23 seconds for 2mm division. Find the error in staff reading at a distance of 100m caused by the bubble out by one division.

Solution : The displacement of tne centre of the bubble = 1 division. The angle through which the line of collimation is moved due to displacement of the bubble,

$$\alpha = 1 \times 23 \text{ seconds} = \frac{23}{206,265} \text{ radians}$$

We know, the angle of inclination in radians, $\alpha = \frac{S}{D}$; where S = error in staff reading and D = distance between the staff and instrument.

$$\therefore \text{ Error in the staff reading} = \text{Distance} \times \alpha = \frac{100 \times 23}{206,265} = \mathbf{0.011m}$$

Example 2. If the bubble tube of a level has a sensitiveness of 35" per 2mm division, find the error in staff reading on a vertical staff at a distance of 90m caused by the bubble bending $1^1/_2$ divisions out of centre. **(A.M.I.E.)**

Solution : The angular value of one division, $\alpha = \frac{S}{Dn} \times 206265$ seconds. Where S = staff intercept or error in staff reading due to deviation of bubble; n = number of divisions through which the bubble is out; and D = distance of the staff from the instrument. Substituting the value in the above equation, we get

$$S = \frac{\alpha\, Dn}{206{,}265} = \frac{35 \times 90 \times 1.5}{206{,}265} = \mathbf{0.023m.}$$

Example 3. A staff held at a distance of 70m from the instrument. If the bubble tube of a level has sensitiveness of 30 seconds per division, find the error in the staff reading when the bubble is 2 divisions out of centre.

Solution : The displacement of centre of the bubble = 2 divisions.

The angle of inclination, $\alpha = 2 \times 30$ seconds $= \frac{60}{206{,}265}$ radians

Now, error in the staff reading = distance x α (in radians)

$$= \frac{70 \times 60}{206{,}265} = 0.02\text{m.}$$

Example 4. A dumpy level set up at a distance of 60.4 metres from the staff records a difference of 0.006m when the telescope is tilted from the horizontal plane and the bubble is away from the centre by one division towards the objective end of the telescope. Calculate the sensitivity of the level. **(L. Min. E. 1985)**

Solution : We know, the angle of inclination of the bubble line

$$= \frac{\text{Error in staff reading}}{\text{Distance of staff from level}} \text{ radians.}$$

$$\text{Angle of inclination} = \frac{0.006}{60.4} \times 206265 \text{ seconds} = 20.5 \text{ seconds.}$$

Hence the sensitivity of the level is 20.5 seconds per division.

Levelling Head : It consists of two tribrach plates each have three arms. They are kept apart to a constant distance by means of levellig screws. These levelling screws are used for bringing the bubble in the centre of its run. It has also an outer hollow conical socket into which fits the inner solid spindle of the telescope thus representing the vertical axis. There is an arrangmement to fix the level over the tripod.

Most modern instruments have three levelling screws. The three-screw arrangement is faster and not subject to rocking that takes place in the four-screw type when two oppostie

screws are turned up or down slightly more than the other two. The disadvantages of the three-screw type is that slight differences in elevation of the line of sight result if all three screws are turned up or all three turned down. Manipulation of the four-screw head does not change the telescope elevation. Also after the threads become worn on a three-screw levelling head, there is some loss of rigidity and the screw must be replaced. Tightening one screw or each pair in the four-screw arangement results in a clamping section and produces a stable set up.

Tripod Stand : When the level is to be used it is fitted over a tripod stand which has external threads on its top. Several types of tripods are available. The legs may be fixed or adjustable in length, and solid or split. All types are shod with metallic conical points with foot rests at lower end and hinged at the top where they connect to a metal head. An adjustable-leg tripod is advantageous for setups in rough terrain but the type with a fixed length may be slightly more rigid. The split leg model is lighter than the solid type but less rugged. A wide-framed tripod, first used on Europian Instruments, is now available from American Manufacturers. A sturdy tripod in good condition is necessary to obtain the best results from the instrument. When the tripod is not in use, it is tied with leather strap and a cap is screwed to its top to protect the threads from injury or wear and tear.

LEVELLING STAFF

A graduated rod used for observing the vertical distances of points above or below the line of sight is known as a levelling staff. Levelling staves are of two types :

1. Self-reading staff, and
2. Target staff.

1. Self-Reading Staff

The staff which can be read directly by the level man as he sights through the telescope and notes the apparent intersection of the horizontal cross-hair with the staff is called a *Self-Reading Staff.* There are various forms of levelling staves made of wood, fibreglass and metal, but the following are the common forms which are used -

(a) Roorke Pattern or one length staff.
(b) Sopwith telescope staff.
(c) Folding staff (as per ISI specification).
(d) Invar precision staff.

(a) Roorke Pattern or One Length Staff : The staff is made from well-seasoned pine or deodar wood. It is usually 3 metres in length, 75mm in width and 25mm to 40mm in thickness. Each metre length is divided into 10 parts, each part is equal to 1 decimetre. One dicimetre is further graduated into 20 divisions, each being of 5mm. The divisions are alternately painted in black and white. Every decimetre length is figured with the corresponding numericals in black. Each metre numerical is red and marked with a letter 'm' (metre) in red on the opposite side. To protect ends of the staff brass heels are provided at the bottom and at the top. The

zero reading on the staff is at the bottom of this heal. The graduated figures are erect and when seen through the telescope they are seen inverted.

Being solid, it is heavy and also due to its one piece length it is difficult to carry. It is liable to bowing. Its long length makes it difficult to hold vertically even in a calm day. It can break.easily on falling but at the same time it can be easily repaired.

(b) Sopwith Telescopic Staff: The most common type of levelling staff is the sopwith, usually 4m in length and divided into three sections, each sliding telescopically into the other. The top portion of length 1.25m is a solid piece of wood and slides into the central wooden box of 1.25m in length. The central box in turn slides into the lower box of 1.5m length, 10cm wide and 6 to 7 cm deep. Each length of the staff when taken out to its full length is held in position by means of a brass spring catch. The graduations are marked erect and hence the readings through a telescope appear to be inverted and above downwards.

(c) Folding Staff: The staff shall be made from a thoroughly seasoned pine or deodar wood 4m in length. The staff comprises of two metres wooden pieces with the joint assembly. The width and thickness of the staff are 75mm and 18mm respectively.

The folding joint of the staff is made of the detachable type with a locking device at the back. When the two portions are locked, it becomes a rigid and staight staff. A circular bubble of 25 minutes sensitivity is attached at the back to errect the staff vertically.

Each metre is divided into 200 divisions, the thickness of the graduations being 5mm. The divisions are alternately painted in black and white. Every decimetre length is figured with the corresponding numerals (the metre in red and decimetres in black). The decimetre marking is continuous throughout the staff. The graduations are marked inverted so that when seen through the telescope they appear erect.

Folding type staff is light in weight, easy to carry and more stable. But it must be handled carefully.

(d) Invar Precision Staff: For every precise work sometimes an invar band graduated upto 5mm interval is fitted to a wooden staff lightly fastened at the lower end and by a spring at the upper end so that any extension in staff may have no influence on the invar band. It eliminates the effect of changes in temperature.

2. Target-Staff

For very precise work and when the sights are long one target staff also known as Philadelphia rod is used. In this type of staff a movable target is provided. The target slides over the staff and consists of a circular disc of about 12.5cm in diameter with horizontal and vertical lines formed by the junction of alternate quadrants of red and white. A rectangular opening in the front of the target exposes a portion of the rod to view so that the reading can be taken. The target is also provided with a vernier. With the target staff, the leveller directs the staff man to slide the target up or down until it is bisected by the line of sight. The target clamped in this position, both the level man and staff man can observe the reading on staff. This type of staff would probably be used by mine surveyors for subsidence levelling, where great accuracy is required.

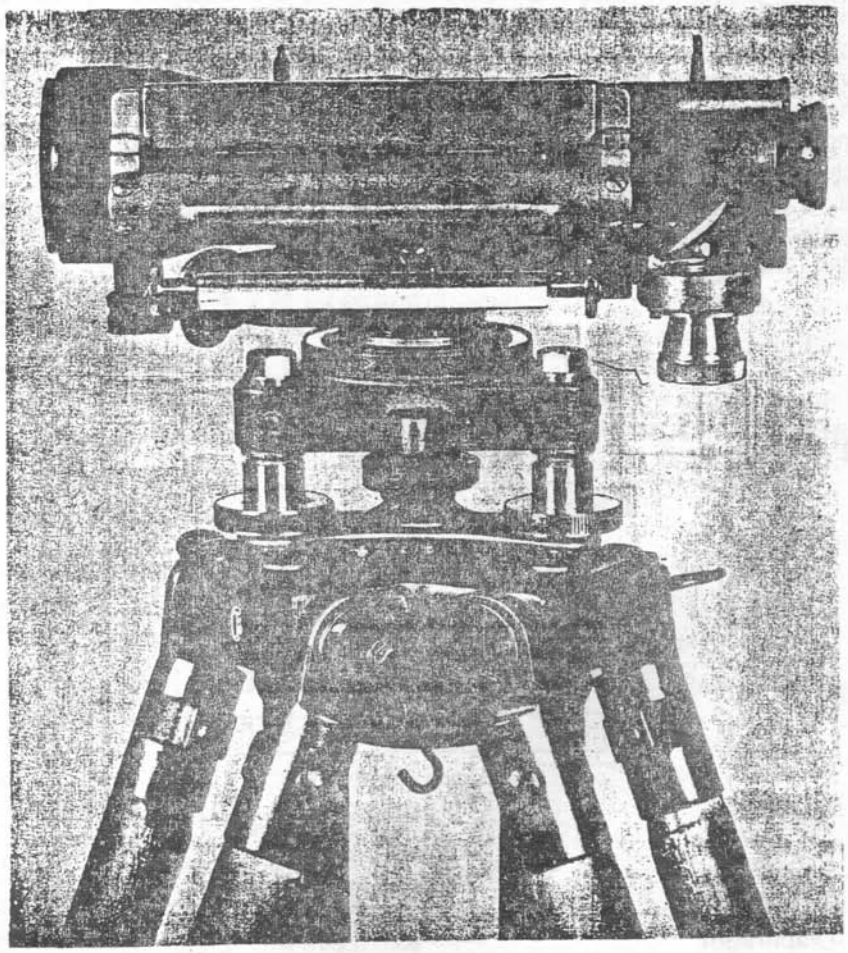
A large staff should not be left leaning against a tree or building or laid on any surface with the graduated face down. Hands must be kept off the painted markings, because worn face make the staff unfit for use.

TYPES OF LEVEL

According to their general arrangement levels may be classified as :

1. Dumpy Level.
2. Wye Level.
3. Cook's Reversible Level.
4. Cushing's Level.
5. Tilting Level.
6. Autoset Level.

Dumpy Level : The instrument is, essentially, a spirit level attached to a telescope which is righidly mounted on to a vertical centre bearing and at right angles to it. It depends for its

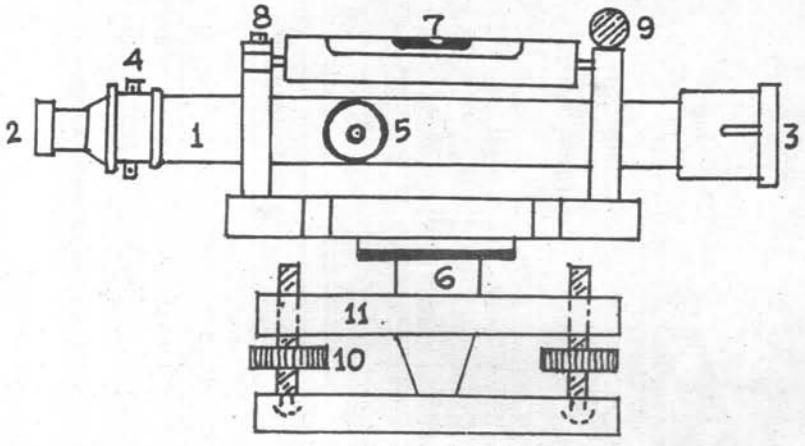


Tilting level (Cooke. Troughton and Simms S 300)

accuracy on the verticality of the axis. The dumpy level may be fitted with a three or four screw levelling arrangement.

The instrument consists of a telescope with an adjustable eye-piece at one end and an object-glass at the other end. The telescope is fitted with an iternal diaphragm near to the eye-piece, and which may consist of cross-hairs or fine lines on glass. The body of the telescope consists of two tubes, one sliding within the other and the movement is controlled by a rack and pinion arrangement, operated by the milled wheel mounted on the side of the telescope. On top or on the side of the telescope a long spirit-level is fitted by means of capstan screws or hinge and adjusting screw which allows for the adjustment of the level with respect to the telescope and vertical axis of the instrument. Usually a small transverse spirit level or a circular spirit-level is attached to the telescope to facilitate rapid levelling of the instrrment.

The telescope tube and the vertical spindle are cast in one piece. The spindle revolves in the socket of the levelling head or tribrach. The solid design gives greater rigidity and stability and reduces the overall height of the instrument. The telescope in some forms of Dumpy level is connected by adjustable straps or collars to a horizontal bar or stage placed beneath the telescope. A quick levelling arrangement based on the principle of ball-and-socket principle to enable the instrment for approximate levelling in a minimum time may also be included in some instrument. The instrument can be accurately levelled by means of three levelling screws. On some older type of levelling arrangement consists of two parallel plates operated by four levelling screws. The lower parallel plate of a dumpy level is bored and threaded for mounting it on its tripod.



Dumpy Level

1. Telescope
2. Eye-piece
3. Object-glass
4. Diaphragm
5. Milled Head (focussing screw)
6. Vertical spindle
7. Spirit Level
8. Adjustable Screw
9. Transverse spirit level
10. Levelling screws
11. Tribrach Stage
12. Trivet Stage

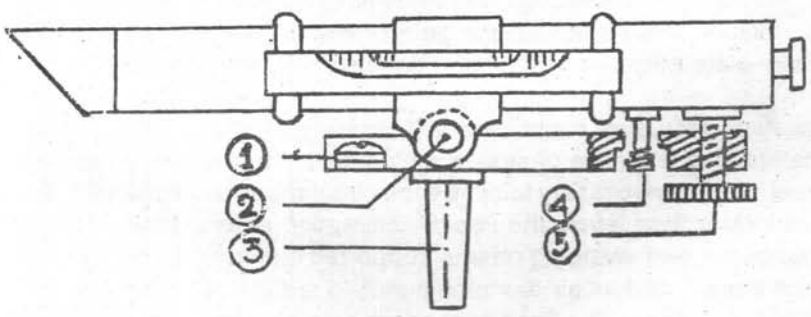
2. Wye Level or Y-level : In this level, the telescope rests on two Y-shaped supports and the telescope is supported in position by two clips which can be fastened to Y-supports by pins. The telescope can be reversed end for end and can be revolved about its longitudinal axis in the Y-supports when the clips are unfastened. The bubble tube is attached at the limb. To the limb plate are attached limb nuts under Y-supports. These are made use in the permanent adjustement. Other parts are similar to a dumpy level.

It has the advantage that the permanent adjustment can be done quickly than a dumpy level, but the adjustment does not stay longer period as it has many loose and movable parts.

3. Cooke's Reversible Level : M/s. Cooke York Company combined good features of dumpy and Y-levels in one by providing a stop flange screw for reversibility of the telescope and named the level as Cooke's Riversible Level. By slackening the stop screw, the telescope can be rotated about its longitudinal axis in the sockets and can also be withdrawn from the sockets and replaced end for end. The disadvantages of this level are (i) the operation of withdrawing the telescope from the socket and replacing it end for end may disturb the position of the instrument, and (ii) it is heavier due to the provision of additional tubes.

4. Cushing's Level : In this level, the telescope is firmly secured to the supports which are fitted to the limb plate. The eye-piece and the object glass are removable and can be interchanged thereby reversing the line of collimation. The end collars are made exactly similar for both eye-piece and object-glass assembly. Slot and pin arrangement is provided for removing and fixing the eye-piece and object-glass. The only disadvantage with this instrument is that it is difficult to fix correctly the eye-piece and object-glass in their positions. Also if the pins and screws are not properly tightened the adjustment will be disturbed.

5. Tilting Level : In tilting levels the line of sight can be tilted slightly without tilting the vertical axis. Thus, the line of sight and the vertical axis need not be exactly perpendicular to



Tilting Level

1. Circular level
2. Tilt axis
3. Azimuth axis
4. Opposing spring
5. Micrometer Screw

each other. A bull's eye (circular) spirit level is a available for quick approximate levelling by means of the levelling screws. Exact level of the instrumnet is obtained by tilting or rotating the telescope slightly in a vrtical plane about a fulcrum of the vertical axis of the instrument without changing the height of the instrument. A micrometer screw acting against a spring under the eye-piece controls this movement.

The tilting feature saves time and increases accuracy, since only one screw need be manipulated to keep the line of sight horizontal as the telescope is turned about a vertical axis. The telescope bubble is viewed through a system of prisms from the observer's normal position behind the eye-piece. The system splits the bubble image into two parts. Centering the bubble is accomplished by making the images of the two ends of air space coincide. It is, however, essential that the observer should have the view of the bubble tube immediately before each observation of the staff.

A tilting level is mainly designed for precise levelling work. It has the advantages that the permanent adjustment of the instrument is not affected by the verticality of the coarse levelling stage, nor even any wear of the levelling mechanism. A horizontal circle in tilting level is a valuable asset since it greatly increases the scope of the instument's utility.

The magnification of the telescope of a *cooke, throughton and Simms 300 Level* is X 25, equivalent focal length 20. 83cm; aparture 1.8cm; stadia interval 1 in 100; distance at which 3mm can be resolved 300m. The sensitivity of the spirit-level is 30"/2mm. The length of the instrument is 19cm; height 11.5cm; weight 2.25 kg; and it fits into a case no longer than 25cm x 12cm x 15cm.

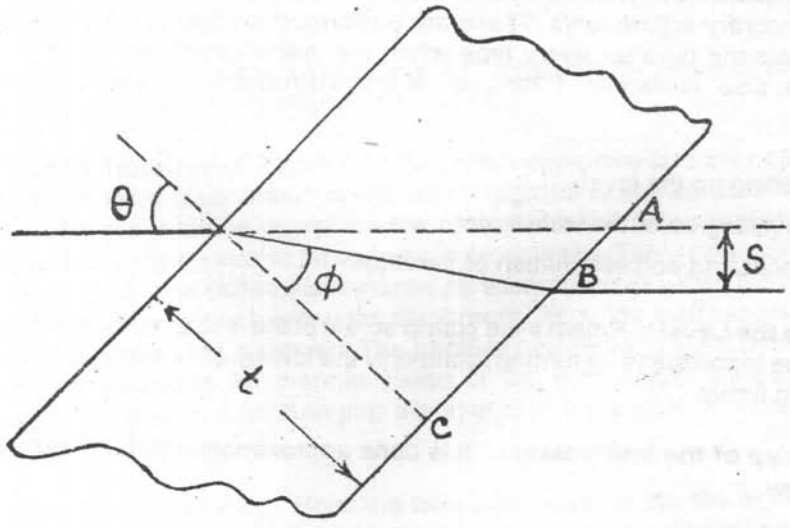
6. Automatic Level : The automatic level also known as *self-aligning levels*, are of recent development and have become very popular for general use because of the operational comfort and rapidity with which they can be operated, high precision, freedom from errors and external influences, and high range of application. The fundamental difference between the automatic level and the classic spirit level is that, in the former the line of sight is no longer levelled manually using the tubular spirit level but levelled automatically. Within a certain tilt range this is achieved by an inclination compensating device called tilt compensator suspended like a pendulum and inserted in the path of light rays through the telescope. When fitted with a parallel plate micrometer the level bocomes a precision level.

The **Watts Autoset** Level contains a self-levelling device, called a stabiliser, which automatically maintains a level line of sight even when the telescope is slightly tilted. When setting up the level it is only necessary to centre the circular bubble attached to the instrument and the optical stabiliser then levels the line of collimation automatically. The stabiliser unit consists of a fixed prism, two swinging prisms supported on four metallic tapes which form a cross-spring flexible pivot, and an air damping piston to reduce oscillations of the prisms and the effect of ground vibrations. The fixed roof prism acts as a reflector and gives an upright image in the eye-piece. The swinging prisms alter their positions according to the tilt of the telescope and automatically maintain the line of sight perfectly horizontal. The magnification of the telescope is x 32; stadia interval 1 in 100; range of stabiliser $\pm$ 20; re-levelling accuracy less than 1" of arc. The overall length of the telescope is 265mm; height of the instrument 130mm and weight about 3 kg.

Carl Zeiss are manufacturing different types of automatic levels they are Ni_1, Ni_2, Ni_{22}, etc. **Zeiss Self-aligning Levels :** Two mirrors are fixed to the barrel of the telescope which the third mirror is suspended by wires from the top of the barrel in these levels. The mirrors have been replaced by prisms to reduce the loss of light. Ni 007 automatic level has been recommended for establishing surface and underground bench marks. It is less affected by refraction disturbances near the ground. **Wild N_3 precision Level** has been adopted universally as the ideal for all precise surveys including deformation surveys and control measurements.

Parallel Plate Micrometer

This device is used for measuring the interval between the line of collimation and the nearest division on the staff to 0.001m directly and 0.0001m by estimation. This type of micrometer, also known as *Optical micrometer*, is a simple device fitted to the precise (tilting) levels such as Watts, Zeiss, etc. It consists of glass parallel sided disc or plate mounted in tubular casing which fits on to the objective end of the telescope. The plate is pivoted centrally so that by turning the micrometer knob it can be tilted about a horizontal axis which is perpendicular to line of sight. When in its vertical position, rays of light pass through it without displacement, but when tilted, refraction causes an apparent shift of the reading hairs over the face of the staff. In this way the line of sight can be made to coincide with the nearest division on the staff, the displacement being a function of the tilting imparted to the parallel plate. The latter is geared, through a tilting drum, to a specially graduated drum or arc which is subdivided into equal parts to represent the line of sight over one complete staff division. The displacement can therefore be read to the nearest thousandth of meter and by estimation to the nearest ten-thousandth. The readings on the micrometer are in no way attached by the distance between the instrument and staff.



Parallel plate micrometer

Theory : Let t = thickness of the parallel plate.
μ = refractive index from air to glass.
S = displacement of the horizontal hair.
θ = angle of rotation of the parallel plate.

$S = AB \times \cos\theta = t(\tan\theta - \tan\phi)\cos\theta$

If θ is small, $S = t(\theta - \phi) = t\theta - t\phi$

But, the index of refraction, $\mu = \dfrac{\sin\theta}{\sin\phi} = \dfrac{\theta}{\phi}$ (nearly)

or, $\phi = \dfrac{\theta}{\mu}$

Hence, the shift $S = t\theta - t\dfrac{\theta}{\mu} = t\theta\left(1 - \dfrac{1}{\mu}\right)$

ADJUSTMENT OF A LEVEL

A level requires two types of adjustments -

1. Temporary adjustemts, and
2. Permanent adjustments.

TEMPORARY ADJUSTMENTS

Those adjustements which are performed at each set up of level prior to taking observaion are called *temporary adjustments.* These are performed to make the axis of rotation vertical and to eliminate the parallax every time when the instrument is shifted and set up in new positions. It is also known as setting up of the instruments and is made in the following steps :

1. Setting up the level.
2. Levelling up of the instrument.
3. Focussing and elimination of parallax.

1. Setting up the Level : Release the clamp screw of the instrument, hold it in the right hand and fix it on the tripod stand by turning round only the lowerpart with the left hand. Then screw the instrument firmly.

2. Levelling up of the instrument : It is done approximately by legs and correctly by the levelling screw.

(a) Levelling by Legs : Bring all the foot-screws in the middle of their run and place the instrument in a desired position at convenient height with the tribrach plate as nearly horizon-

tal as possible. Fix any two legs firmly into the ground by pressing them with the hand and turn the telescope to be nearly parallel to the line joining the feet of these two legs. Then move the third leg to right or left and in or out to bring the long and corss-bubbles respectively in their central positions. Much time is saved if nearly all the levelling is done by the tripod legs.

(b) Levelling by Foot-screws : Place the telescope parallel to any pair of the foot-screws and bring the long bubble to the centre of its run by turning these screws equally either both inwards or both outwards. To move the bubble to the right, turn the screws inwards and to move it to the left turn the screws outwards *(right in and left out)*. Now move the telescope through 90° clockwise so that it lies over the third foot-screw and bring the bubble in the centre of its run by this third foot-screw applying the same rule *Right in and Left out.*

Bring the telescope back to its original position and bring the bubble in the centre if not. Now again turn through 90° to bring the bubble in the centre. Repeat these operations until the bubble remains centre of its run in both positions. If the instrument is in permanent adjustment the bubble will remain central for all positions of the telescope, if not the instrument needs permanent adjustment.

Note : (i) The instrument should, as far as possible, be set up on a solid ground to avoid its settlement in the course of observations in a setting. If such stable ground is not available, the tripod legs should be pressed firmly into the ground.

(ii) While setting the instrument on a pucca floor, the shoes of the tripod should, as far as possible, be placed in the joints to prevent the legs from spreading.

(iii) When setting on sloping ground, two legs should be kept down slope and third up slope.

(iv) Before the instrument is set up at a new place, the foot-screws should be brought to the centre of their run and also the micrometer screw is to be set so that it should read zero on both the scales.

(v) The limit of distance for clear visibility of staff by an ordinary level is 100 metres and in case of tilting levels it varies from 300 to 350 metres.

3. Focussing and Elimination of Parallax : To set the eye-piece and the object-glass at a proper distance apart for a clear vision of the object sighted is known as focussing. If the diaphragm is out of focus with the eye-piece the cross hairs will be seen distinctly and the image of a distant object will appear to flit accross to cross-hairs. This apparent movement of image relative to cross-hairs is called *instrumental parallax.* It occurs when the image formed by the object-glass falls short of or beyond the diaphragm. Accurate staff readings can not be taken until the parallax has been removed. The parallax can be eliminated in two steps viz.
(i) Focussing the eye-piece for distinct vision of the cross-hairs at diaphragm, and
(ii) Focussing the object-glass for bringing the image of the object into the plane of the diaphragm.

(i) Focussing the Eye-piece : Point the telescope towards the sky or hold a piece of paper in front of the telescope. Move the eye-piece in and out until a distinct and sharp black image of the cross-hairs is seen.

(ii) Focussing the Object-glass : Direct the telescope towards the staff and turn the focussing screw until a clear and sharp image of the staff graduations is obtained. If there is no change in the staff reading when the observer's eye is moved up and down, the parallax is completely eliminated.

PERMANENT ADJUSTMENT

Permanent adjustments of levels consist in establishing a fixed relationship between the fundamental lines of the levels. For accurate levelling the instrument must be in adjustment. The following are the fundamental lines of a levelling instrument :

1. The axis of the bubble tube.
2. The line of collimation.
3. The vertical axis.
4. The axis of the telescope.

Relationship between these lines :

(a) The line of collimation should be parallel to the axis of the bubble tube.

(b) The line of collimation should coincide with the axis of the telescope.

(c) The axis of the bubble tube should be at right angles to the vertical axis.

Principle of Reversal

The principle undrlying the method is that if any error exists in a certain part of the instrument, it is doubled by reversing (i. e. revolving through 180°) the position of the part. In other words the magnitude of apparent error on reversal becomes twice the actual error.

For example take the case of a set square, the two sides of which have an error α in perpendicularity. On reversing the square the apparent error becomes 2α.

Most of the adjustments of the level and the theodolite are based on this *Principle of reversal.*

Note : As the use of other levels is becoming obsolete, the parmanent adjustments of Dumpy Level and Tilting Level are explained.

PERMANENT ADJUSTMENT OF A DUMPY LEVEL

In a dumpy level the telescope is rigidly fixed to the spindle. Therefore parts requiring adjustments are the cross hairs and the level tube. The adjustments are :

1. *To make the axis of the bubble tube perpendicular to the vertical axis* (Known as **Bubble adjustment**).
2. *To make the line of the collimation parallel to the axis of the bubble tube* (Known as **Collimation adjustment**).

1. To make the axis of the bubble tube perpendicular to the vertical axis.

Object : The object of this adjustment is to make the vertical axis truly vertical and to ensure that once the temporary adjustment of the instrument is done, the bubble should remain in the centre of its run for all positions of the telescope.

Necessity : If this is not done much time is wasted in bringing the bubble in the centre of its run as it is necessary before taking each reading. Moreover there will be no true difference of level between any two points in which case by over look, the bubble is not brought in the centre and the reading taken. So the adjustment is necessary for convenience and for accurate levelling.

Test; (i) Set up the level to be tested on a firm ground and perform the temporary adjustment.

(ii) When the telescope is on third screw and the bubble is central, rotate the telescope through 180°, say clockwise.

(iii) If the bubble remains still central the adjustment is correct. If not, it needs adjustment of the bubble tube.

Adjustment :

(i) Note the deviation of the bubble from the centre, say 2n divisions. Bring the bubble half way back, *i. e. 'n'* divisions by the third foot screw.
(ii) The remaining half is removed by raising or lowering the one end of the level tube by truning the capstan headed nuts provided at one end of the level tube.
(iii) Turn the telescope through 90° anticlockwise. The telescope will be now parallel to a pair of foot screws. Bring the bubble in the centre of its run by these foot screws *(by Right-in Left-out motion)* if not central.
(iv) Again turn the telescope through 90° clockwise and see whether the bubble remains central or not. If not, repeat the steps (ii) and (iii). Three or four trials are necessary.
(v) As a final check the bubble should remain central for all positions of the telescope when the bubble tube is finally adjusted.

2. To make the line of collimation parallel to the axis of the bubble tube.

Object : The object of this adjustment is to set the line of collimation also at right angles to the vertical axis when the bubble is centered.

Necessity : When the level is centered, the line of collimation will become horizontal and thus all the readings taken will be correct regardless of the direction in which the telescope is pointed.

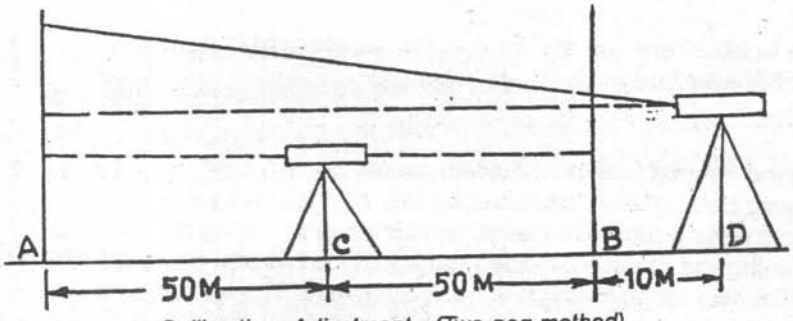
Test and Adjustment

The collimation error is tested and adjusted by the following method known as *Two-peg method.*

Test : (i) Set two pegs A and B on a fairly level ground 100m apart. Set up the level at C exactly midway A and B and do the temporary adjustment.

(ii) Take staff readings by keeping the staff first at A and then at B, taking care that the bubble should remain central while taking the readings, Let the staff readings A and B are h_1 and h_2 respectively. Find out the difference in level between the two points. It will be true difference as the level is set midway even if the line of sight is inclined.

(iii) Shift the level at a point D in line AB about 10 metres away from B. After doing the temporary adjustment again take staff readings at A and B with the bubble central. Let the staff readings on A and B are h_3 and h_4 respectively. Calculate the appartment difference of level.



Collimation Adjustment (Two peg method)

(iv) If the difference found in step (i) tallies with the difference in level of step (ii), the instrument is in adjustment, if not it means that the line of collimation is inclined and needs adjustmnet.

Adjustment

(i) The true difference of level = $h_1 - h_2$ or $h_2 - h_1$ as the peg A is lower than peg B or the peg A is at a rise than peg B. Thus find whether the peg A at a fall or at a rise.

(ii) Now find out the reading on the far peg A by adding or subtracting the true differ ence in the reading of peg B (taken when the level was set at D) as the peg is at a fall or at a rise. Let the readeng be h_5 a

$\therefore h_5 = h_4 \pm$true difference (+ ve sign for fall and - ve sign for rise)

(iii) Compare the calculated reading h_5 with the observed one h_3, if h_3 is greater than h_5 the line of collimation is inclined upwards and if h_3 is less than h_5 the line of collimation is inclined downward. The collimation error thus will be equal to either $h_5 - h_3$ or $h_3 - h_5$ in a distance AB = 100 metres.

(iv) Now find the correction to be applied to the readings on both the pegs

$$\text{per m length} = \frac{(h_5 - h_3) \text{ or } (h_3 - h_5)}{100}$$

$\therefore$ Correction to the reading on the far peg A

$$e_1 = \frac{(h_5 - h_3) \text{ or } (h_3 - h_5)}{100} \times (100 + 10)$$

Correction to the reading on the near peg B

$$e_2 = \frac{(h_5 - h_3) \text{ or } (h_3 - h_5)}{100} \times 10$$

(v) The above corrections are additive to or subtractive from the respective observed staff readings h_3, h_4 (taken when the level was at D) according as the line of collimation is inclined downwards or upwards.

$\therefore$ The correct reading on the peg A = $h_3 \pm e_1$

The correct reading on the peg B = $h_4 \pm e_2$

The adjustment is usually done on the far peg and checked on the near peg.

(vi) Look through the telescope (while the level is at D) the staff kept at farthest point A and adjust the central horizontal cross-hair of diaphragm by moving the diaphragm screws.

For doing this the rule of 3 Ls should be followed. That is Less-loose-lower. If the reading is to be reduced or lessened, loosen the lower screw and tighten the upper one and vice versa.

Thus if the correct reading is greater **(the line of collimation is inclined down wards)** than the observed one, loosen the upper screw and tighten the lower screw; but if the correct reading is less (the line of collimation is inclined upwards) than the observed one, loosen the lower screw and tighten the upper screw so that the horizontal cross-hairs should just touch the correct reading.

(vii) Now check the adjustment by reading the staff kept at the nearest peg B. If the cross-hair is adjusted the reading will tally with the calculated correct reading at B, if not repeat the process until perfect.

Check

(viii) Now set the instrument any where between the pegs A and B and after doing temporary adjustment take readings on the staff held on A and B with bubble central. Find the difference of level. This difference of level should tally with the true difference if the instrument is in permanent adjustment.

Example 5. A dumpy level was set up at C exactly midway between two pags A and B 100m apart. The readings on the staff when held on the pegs A and B were 2.250 and 2.025 respectively. The instrument was then moved and set up at a point D on the line of BA produced, and 20m from A. The respective staff readings on A and B were 1.875 and 1.670. Calculate the staff readings on A and B to give a horizontal line of sight.

Solution : *When the instrument is at C :*

True difference in elevation between A and B
= 2.250 – 2.025 = 0.225m, B being higher.

When the instrment is at D :

Apparent difference in elevation between A and B
= 1.875 – 1.670 = 0.205m, B being higher.

Since the apparent difference of level is not equal to the true difference, the line of collimation is out of adjustment.

∴ The inclination of the line of sight in the net distance AB will be

$$\tan \alpha = \frac{0.225 - 0.205}{100} = \frac{0.020}{100}$$

Since apparent difference of level is lesser, the line of sight is inclined upwards

∴ Correct staff reading at A = 1.875 – AD tan α

$$1.875 - \frac{20 \times 0.020}{100} = \mathbf{1.871}$$

and correct staff reading at B = 1.670 – DB tan α

$$= 1.670 - \frac{120 \times 0.020}{100} = \mathbf{1.646}$$

Example 6. While checking a dumpy level the following readings were obtained :

Level midway between two staff stations A and B 100m apart. Staff readings on A = 1.90m and on B = 1.40m.

Level set up 10m behind B and in line AB. Staff reading on B = 1.10m and on A = 1.35m. Determine the amount of error. Calculate the correct staff readings on A and B to give a horizontal line of sight. **(A.M.I.E. 1977)**

Solution : *When the instrument was set midway -*

True difference of level between A and B = 1.90 – 1.40 = 0.50m fall from B to A.

When the instrument was at C 10m behind B -

Apparent difference of level between A and B = 1.35 – 1.10 = 0.25m.

Since the apparent difference of level is not equal to the ture difference, the line of collimation is out of adjustment. When the instrument was at C, the corresponding reading at A should be = 1.10 + 0.50 = 1.60m (A being fall). But the observed reading at A = 1.35m is less than 1.60m, the line of cllimation is inclined upwards.

Collimation error in 100m length = 1.60 – 1.35 = 0.25m

Error per metre length = $\frac{0.25}{100}$ m.

Error in 110m length = $\frac{0.25}{100}$ x 110 = **0.275m**

Error in 10m length = $\frac{0.25}{100}$ x 10 = **0.025m**

Hence correct reading at A = 1.35 + 0.275 = **1.625m**
and correct reading at B = 1.10 + 0.025 = **1.125**

Check : Correct difference of level = 1.625 – 1.125 = 0.500m.

Example 7. A Dumpy Level set up at a mid position between two pegs P and Q, 100 metres apart gave the following readings :

P – 1.23m Q – 1.47m

The same Dumpy level set up at a distance of 15 metres away form P in the same line gave the following readings :

P – 1.09m Q – 1.51m

What may be the reason for this discrepancy ? How do you rectify such maladjustments ?

Solution : *When the instrument was midway between P and Q :*

True difference in level = 1.47 – 1.23 = 0.24 (P at higher level)

When the instrument was at a distance 15m away from P :

Apparent difference of level between A and B = 1.51 – 1.09 = 0.42m.

Since the apparent difference of level is not equal to true difference, the line of collimation is out of adjustment. The corresponding reading at Q in second setting of the instrument should be 1.09 + 0.24 = 1.33m (Q being fall). But the observed reading at Q was 1.51 is more than 1.33m. The line of collimation is, therefore, inclined upwards.

Collimation error over a distance of 100m = 1.51 – 1.33 = 0.18m (+ ve).

To rectify the maladjustment the following corrections would be required.

Correction to staff reading on Q = $\frac{0.18}{100}$ x (100 + 15) = 0.207m (– ve).

Correction to staff reading on P = $\frac{0.18}{100}$ x 15 = 0.027m (– ve).

The **correct reading at Q** in second setteing of the instrument should be (1.51 – 0.207) = 1.303m and that **at P** should be (1.09 – 0.027) = **1.063m.**

Since the correct redings are less than the observed readings, the cross-hair has to be raised from its true position by loosening the lower capstan screw which secure the diaphragm and tightening the upper one till the intersecteion coincides with the correct staff reading at far station Q first and then at P.

[**Remember** the *rule of three Ls* **Less Loose Lower** i. e. if the reading seen through the telescope is to be lessened, then **Loosen** the lower screw and **tighten** the upper one and vice-versa.]

Example 8. The following observations were taken during the testing of a dumpy level :

Instrument at	**Staff reading on**	
	A	**B**
A	1.275	2.005
B	1.040	1.660

Is the instrument in adjustment ? To what reading should the line of collimation be adjusted when the instrument was at B ? **(Survey Comp. 1984)**

Solution : ***Level at A :***

Apparent difference of level between A and B = 2.005 – 1.275 = 0.730m

Level B :

Apparent difference of level between A and B = 1.660 – 1.040 = 0.620m

Hence, the instrument is not in adjustment and the true difference of level

= $\frac{0.730 + 0.620}{2}$ = 0.675m fall from A to B

When the instrument was at B -

The correct staff reading at B = 1.660

The correct staff reading at A = 1.660 – 0.675 = 0.985m.

The line of collimation is inclined upwards, the correct reading being less than the observed one, and the error = 1.040 – 0.985 = 0.055m. This error should be adjusted to read the staff at A 0.975m when the instrument at B giving a reading 1.660m on its nearer staff.

Example 9 : Two pegs A and B were driven 80m apart. A dumpy level was set near A and the staff readings taken on A and B were 1.630 and 1.645 respectively. The level was then shifted and placed near B and the respective staff readings were 1.705 and 1.690. State whether the instrument is in adjustment or not. Find the R.L. of A, if that of B is 70.612m.

Solution : Apparent difference of level between A and B in the first case = 1.645 – 1.630 = 0.015m (fall from A to B).

Apparent difference of level between A and B in the second case = 1.705 – 1.690 = 0.015, (rise from A to B).

Since one set up of the instrument indicates fall from A to B and the other rise from A to B, the line of collimatron **is not in adjustment.**

$$\text{True difference of level between A and B} = \frac{0.015\ (\text{fall}) + 0.015\ (\text{rise})}{2}$$

$$= \frac{-0.015 + 0.015}{2} = 0$$

This shows that the pegs A and B are at the same level and hence the reduced level of A = **70.612m.**

Example 10. To find R.L. of a point B on the top of a hill, flying levels were run from a bench mark A of R.L.245.00m. The average length of the ten back-sights and the ten fore-sights were found to be 30m and 20m respectively and the difference in elevation of A and B was observed to be 21.650m. The instrument was tested for checking the collimation error and the following results were obtained:

Level at	Readings on		Remarks
	P	Q	
M	1.598	2.220	M is midway between P and Q.
N	1.755	2.382	(PQ = 100m) N is on the line QP Produced and 20m from P.

Calculate the R L. of B.

Solution : *Level at M :*

True difference of level between P and Q = 2.220 – 1.598 = 0.622m fall from P to Q.

Level at N :

Apparent difference of level between P and Q = 2.382 – 1.755 = 0.627m fall from P to Q.

The line of collimation is not in adjustment. Now the reading at P in second setting + true fall = 1.755 + 0.622 = 2.377m should be the correct reading at Q in second setting of the instrument, But the observed staff reading 2.382 which is more than the true value (2.377), the line of collimation is there fore inclined upwards. The collimation error = 2.382 – 2.377 = + 0.005m in 100m.

The total length in backsights	= 10 x 30	= 300m
The total length in foresights	= 10 x 20	= 200m
Total error in backsights	= $^{300}/_{100}$ x 0.005	= 0.015m
Total error in foresights	= $^{200}/_{100}$ x 0.005	= 0.010m
Net error is therefore	= 0.015 – 0.010	= 0.005m (+ ve).
Hence the true difference of level	= 21.650 – 0.005	= 21.645m

∴ R. L. of B = 245.000 + 21.645 = **266.645 m.**

Example 11. Two points A and B are selected at a distance of 50 metres apart from each other. A Dumpy Level was set up with its eye-piece almost against the staff at A and then B and the following readings were noted :

Staff reading on A – 1.429m
Staff reading on B – 1.847m
} Level at A

Staff reading on B – 1.666m
Staff reading on A – 1.354m
} Level at B

Calculate how much adjustments will be necessary for the cross-hair to make the line of sight parallel to the axis of the bubble.

Solution : *Level at A :*

Apparent difference of level between A and B = 1.847 – 1.429 = 0.418m
(fall from A to B.)

Level at B :

Apparent difference of level between A and B = 1.666 – 1.345 = 0.312m
(fall from A to B)

Since the two reslults differ, the instrument is out of adjustement. The true difference of level between A and B = $\frac{0.418 + 0.312}{2}$ = 0.365m fall from A to B.

When the instrument was at B-

The correct staff reading on B = 1.666m.

The correct staff reading on A = 1.666 – 0.365 = 1.301m.

The line of collimation is inclined upwards, the correct reading should be less than the observed one. The error = 1.354 – 1.301 = 0.053m (+ve). This error should be adjusted to read the staff on A 1.301m when the instrument at B giving a reading 1.666m on staff at B.

Example 12. In testing a dumpy level for collimation error, the staff reading on two pegs, when the instrument was midway between them, were 2.685 and 2.595, the distance between the pegs 60 metres. The staff readings given by the instrument, when moved to a position 12 metres beyond the higher peg were 2.515 and 2.350. What was the collimation error of the instrument ? To what reading must the instrument be set on the nearer staff in oder to place the line of collimation truly horizontal ?

Solution : At first setting of level -

True difference in level = 2.685 – 2.595 = 0.090m.

At second setting of the level -

Apparent difference in level = 2.515 – 2.350 = 0.165m.

Collimation error = 0.165 – 0.090 = 0.075 (+ve)

This error is over a distance of 60 metres and, therefore, the error at 12 metres

$$= \frac{0.075 \times 12}{60} = 0.015\text{m.}$$

The reading on the nearer staff should be : 2.350 – 0.015 = **2.335 metres.**

PERMANENT ADJUSTEMENT OF A TILTING LEVEL

In tilting level, the telescope and the bubble tube attached to it can be tilted by means of tilting screw or micrometer drum independently of the vertical axis. Therefore only condition of adjustment of tilting level is -

The line of collimation should be parallel to the axis of bubble tube.
(Known as collimation adjustment).

Object : The object of this adjustment is to ensure that the line of sight should rotate in the horizontal plane as long as the bubble remains central.

Necessity : If this is not done, and the level is not placed midway, the true difference of level obtained between any two consecutive staff readings is incorrect. So it is essential for convenience and for accurate results.

Test : Any method as described for the adjustment of Dumpy Level can be used.

(i) Set up the level midway two pegs A and B fixed on a smooth ground 60 to 100m apart. Do the temporary adjustment of the level and take the staff readings on A and B with bubble central by means of the tilting screw. Find the true difference of level.

Note : Before setting the level bring the micrometer drum zero to coincide with zero of the vertical scale.

(ii) Shift the level just near to one of the pegs say A and again do the temporary adjustment. With the bubble brought central by means of the micrometer drum take staff readings on both of pegs A and B. Find the apparent diference. If true difference tallies with this difference, the instrument is in adjustment, if not the level requires adjustment.

Adjustment : (i) Considering the staff reading at peg A correct being close to level find the correct reading at peg B by adding or subtracting the true difference as the peg B is at a fall or at a rise. Also find whether the line of collimation is depressed below or inclined upwards.

(ii) Hold the staff at the farthest peg B and set the telescope to the corrected staff reading by means of a tilting screw. It will be seen that the bubble of the long level tube is moved from its central position.

(iii) Bring the buble of this tube to the centre of its run by means of capstan headed nuts provided at one end of the level tube.

(iv) If the micrometer screw zero is not coinciding with the zero of the vertical scale, loosen the small screw provided on the micrometer drum and rotate it up or down so that the zero of the horizontal scale should coincide with the zero of the vertical scale. Now tighten the small screw.

(v) Repeat the process until correct.

REDUCTION OF LEVELS

LEVEL BOOK : The levelling field-notes are entered in a note book Called a level book. The pages of a level-book are printed in a tabular form so as to facilitate booking of readings and reduction of levels.

REDUCTION OF LEVELS : There two methods of working out the reduced levels of the points from the staff readings taken in the field -

1. *Height of instrument or Collimation method.*
2. *Rise and Fall method.*

1. Collimation Method : The "height of collimation" is the height of the line of sight of the telescope above datum. The collimation method consists in finding the elevation of plane of collimation or height of instrument (H.I.) for every setting of the instrument and then obtaining the reduced levels of the points with reference to the respective plane of collimation. To start with, the H. I. is calculated by adding the R.L. of the back sight of the starting point. The reduced level of the intermediate points and the first change point are then obtaind by subtracting the respective readings from the H.I. When the instrument is shifted, a new plane of collimation is set up and the height of which is calculated by adding the back sight reading to the R.L. of the first change point. The reduced level of the successive points and the second change point are found out by subtracting their staff readings from this new H.I. The process is repeated until all the reduced levels are worked out, and then the arithmetical check is applied.

Check : The difference between the sum of the back sights and the sum of the fore sights should be equal to the difference of the first and the last R. Ls. i.e. Σ BS – Σ FS = Last R.L – First R.L. But if an error has been made in calculating the reduced level of an intermediate staff station, the error is not carried forward and therefore, not revealed in this check. **The intermediate levels can, however, be checked** as follows -

The sum of all reduced levels except the first should equal the sum of each instrument height multiplied by the number of fore and intermediate sights taken from it minus the sum of the fore and intermediate sights readings.

A typical page of bookings using the collimation method is shown below :

B. S.	I. S.	F. S.	Height of Collimation	Reduced Level	Distance (metres)	Remarks
1.025			61.025	60.000	0	B. M.
	1.700			59.325	50	
2.505		1.915	61.615	59.110	100	Change point
	1.805			59.810	150	
	2.015			59.600	200	
0.400		2.760	59.255	58.855	250	Change point
		1.590		57.665	300	
3.930	5.520	6.265		60.000		
		3.930		57.665		
		– 2.335		– 2.335		

Referring to the page of levelling above, this would be -
The sum of reduced levels except first = 354.365.

Sum of cach instrument height x No. of fore and intermediate sights taken from it :
= (61.025 x 2) + (61.615 x 3) + (59.255 x 1) = 122.050 + 184.845 + 59.255 = 366.150.
Sum of fore and inermediate sight readings = 6.265 + 5.520 = 11.785.
Therefore, the formula reads : 354.365 = 366.150 – 11.785 = 354.365 and calculation of the readuced levels has been carried out correctly.

Advantages and disadvantages : The collimation method provides considerable saving of labour as this method dispenses with finding rises and falls in the other method. It is less tedious, more rapid and involves less calculations. The error in calculating the R.L. of any point is not carried forward because the R.Ls are calculated from the height of collimation. The main advantage is when a large number of intermediate sights, have to be observed from each setting of the instrument. Another application of this method may be where certain points have to be set out at specified levels above datum. In this case it is very simple matter to calculate the required staff readings to fix a particular reduced level. The system is generally used in setting out for constructional works.

2. Rise and fall Method : In this method, the difference between consecutive points is calculated by comparing each point after the first with that immediately preceding it. If a staff reading (fore reading) is greater than at the preceding point, then there is a fall. On the other hand if the staff reading of the following point is smaller than the preceding point, the difference in reading is a rise.

The reduced level of each point is then found out by adding rise or subracting fall to or from the R.L. of the preceding point.

Arithmetical Check : In this method, there are three checks on the accuracy of levels. The difference between the sum of the back sights and the sum of the fore sights is equal to the difference between the sum of the rises and that of the falls is equal to the difference between the first and the last R Ls. i.e.

$$\Sigma \text{ B.S.} - \Sigma \text{ F.S.} = \Sigma \text{ Rise} - \Sigma \text{ Fall} = \text{Last R.L.} - \text{First R.L.}$$

If the difference is positive (+ve) it shows that the following ground is at a rise and if it is negative (–ve), the following ground is at a fall.

A typical page of bookings using the Rise and Fall method is shown below :

Station	Dist	Bearing	Readings			Rise	Fall	Reduced Level	Remarks
			B. S.	I. S.	F. S.				
1			1.650					100.530	Bench Mark
2			2.125		1.855		0.205	100.325	Change Point
3			2.625		2.420		0.325	100.000	Change Point
4			1.985		2.755		0.130	99.870	Change Point
5			2.110		1.885	0.100		99.970	Change Point
6				2.205			0.095	99.875	
7					2.455		0.250	99.625	Last Point
	Total		10.495		11.400	0.100	1.005		

Arithmetical check :

Σ B. S. – Σ F. S. = Σ Rise – Σ Fall = Last R.L – First R.L.

10.495 – 11.400 = 0.100 – 1.005 = 99.625 – 100.530

= – 0.905 = – 0.905 = – 0.905.

Advantages and Disadvantages : The method has three checks for arithmetical accuracy. But it is more laborious as it involves more calculation work in finding rise and fall of each point. The mistake done in calculating the R.L. of one point will be carried forward.

The rise and fall method is used for calculating precise levelling operations, and for earth work calculation work.

INVERTED-STAFF READING : Occassionally it is impossible to hold the staff upright on a point at which a reading is required. If practicable, the staff may be inverted and its base held on the required point. A reading to a staff held in this position is entered in the level-book in brackets or with a negative (–ve) sign. If the rise and fall method of booking is used in working out rises and falls an inverted-staff reading must always be added to the preceding normal reading and the amount entered in the rise column and an inverted-staff reading must always be added to the succeeding normal reading and the amount entered in the fall column. If the height of collimation method of booking is used, then all inverted-staff readings are simply added to the height of collimation of the instrument.

Rise and Fall Method of Booking

B. S.	I. S.	F. S.	Rise	Fall	Rdeuced Level (m)	Distance (m)	Remarks
1.435					149.650	0	Rail level at 20's junction
	– 2.585		4.020		153.670	9	Height of roof.
	– 2.540			0.045	1.53.625	24	
1.925		1.370		3.910	149.715	30	
	– 2.260		4.185		153.900	45	
	1.295			3.555	150.345	45	
	– 2.635		3.930		154.275	60	
	1.250			3.885	150.390	60	
	– 2.465		3.715		154.105	75	
	1.375			3.840	150.265	75	
	– 2.290		3.665		153.930	90	
	– 2.315		0.025		153.955	120	
		1.260		3.575	150.380	129	
3.360		2.630	19.540	18.810	150.380		
2.630			18.810		149.650		
0.730			0.730		0.730		Check

Height of Collimation Method of Booking

B. S	I. S.	F. S.	Height of Collimation	Reduced Level (m)	Distance (m)	Remarks
1.435			151.085	149.650	0	Rail level at – 20's junction
	– 2.585			153.670	9	Height of roof.
	– 2.540			153.625	24	
1.925		1.370	151.640	149.715	30	
	– 2.260			153.900	45	
	1.295			150.345	45	
	– 2.635			154.275	60	
	1.250			150.390	60	
	– 2.465			154.105	75	
	1.375			150.265	75	
	– 2.290			153.930	90	
	– 2.315			153.955	120	
		1.260		150.380	120	

READING ON STAFF NOT HELD VERTICAL : If the staff is not vertical, the reading taken will be too great. The error varies directly with the magnitude of the staff reading and directly as the square root of the inclination. In running a line of levels uphill, backsight readings are likely to be increased more than foresight from this source and the elevation of a bench mark on top will oe too great. Similarly, the elevation of a benchmark at the bottom, when levelling downhill, will be too small. The error, which is positive and systematic in nature, can be minimised by carefully plumbing the staff either by eye estmation, or by using a staff level or by waving the staff slowly and taking the minimum staff reading. Vertical cross-hair may be used to check the verticality of the staff in the direction of traversing to the line of sight.

Example 13. Find the error of reading of a level staff if the observed reading is 3.790m, the staff is 15cm off the vertical through the bottom.

Solution : Correct reading = $\sqrt{(\text{Observed reading})^2 - (\text{distance off the vertical})^2}$

$= \sqrt{3.790^2 - 0.15^2} = \sqrt{14.3641 - 0.0225} = \sqrt{14.3416} = 3.787\text{m}$

Hence, error of reading = 3.790 – 3.787 = **0.003m.**

Example 14. The observed staff reading on a staff held at A was 2.625 metres. The staff was found to be 15 cm off the vertical through its bottom. Find the correct staff reading.

Solution : Observed staff reading = 2.625m

Correct staff reading = $\sqrt{2.625^2 - 0.15^2}$ = 2.6207m

Example 15. The staff reading on a certain peg is 3.280 m when the staff is held 0.25m out of the plumb in its total length of 4m. Find the correct reading for the staff held vertical.

Solution : The staff is held 0.25m out of plumb for its total length of 4m.

At the point, where reading has been observed, it is out of plumb by

$$0.25 \times \frac{3.280}{4} = 0.205\text{m}.$$

The correct reading = $\sqrt{(3.280)^2 - (0.205)^2}$ = 3.274m

Example 16. The following consecutive readings were taken with a level and a 4 metres levellling staff on continuously sloping ground at a common inteval of 30m :

0.585 on A, 0.935, 1.955, 2.845, 3.645, 3.940.

0.960, 1.035, 1.700, 2.535, 3.850, 0.955, 1.580, 3.015 on B.

The elevation on A was 520.450m. Make up a level-book and apply the usual checks. Determine the gradient of the line AB.

Solution : Since the length of the staff is 4 metres, it means the instrument has to be shifted after the staff readings 3.940 and 3.850 as the ground is uniformly sloping. Reading 0. 585 on A will be a backsight, and the readings 3.940, 3.850 and 3.015 will be fore sights. The readings 0.960 and 0.955 will be back sights on change points. All other readings will be intermediate sights.

The reduced levels are calculated by height of instrument method and the result is worked out in table -

Stn.	Dist. m	Bearing	B. S.	I. S.	F. S	Height of Instrument	R.L	Remarks
1	0		0.585			521.035	520.450	B. M. on A
2	30			0.935			520.100	
3	60			1.955			519.080	
4	90			2.845			518.190	
5	120			3.645			517.390	
6	150		0.960		3.940	518.055	517.095	Change Point
7	180			1.035			517.020	
8	210			1.700			516.355	
9	240			2.535			515.520	
10	270		0.955		3.850	515.160	514.205	Change Point
11	300			1.580			513.580	
12	330				3.015		512.145	On B
			2.500		10.805			

Arithmetical Check : Σ B. S. – Σ F. S. = 2.500 – 10.805 = – 8.305

Last R. L – First R. L. = 512.145 – 520.450 = – 8.305

Negative sign indicates fall from A to B and this fall of 8.305m is in a distance of 330 metres.

Hence the **gradient of the line AB** = 8.305 in 330 = **1 in 39.74.**

Example 17. The following consecutive readings were taken with a dumpy level :
0.894, 1.643, 2.896, 3.016, 0.954, 0.692, 0.582, 0.251, 1.532, 0.996, 2.135.

The instrument was shifted after the fourth and the eighth readings, The first reading was taken on the staff held on the bench mark of R. L. 820.765. What is the difference of level between the first and the last points ? **(Survey Comp. 1987)**

Solution : The staff readings are entered in the level field book and R. Ls of the points are calculated :

B. S.	I. S.	F. S.	Rise	Fall	R. L.	Stn.	Remarks
0.894					820.765	1	R. L. of Station 1
	1.643			0.749	820.016	2	= 820.765
	2.896			1.253	818.763	3	
0.954		3.016		0.120	818.643	4	
	0.692		0.262		818.905	5	
	0.582		0.110		819.015	6	
1.532		0.251	0.331		819.346	7	
	0.996		0.536		819.882	8	
		2.135		1.139	818.743	9	
3.380		5.402	1.239	3.261	820.765		
		3.380		1.239	818.743		
		2.022		2.022	2.022		Check

Difference of level between first and last points = 820.765 – 818.743 = **2.022**m.

Example 18. The following consecutive readings were taken with a dumpy level :
0.895, 1.645, 2.895, 3.015, 0.955, 0.695, 0.585, 0.250, 1.535, 0.955, 2.135.

The instrument was shifted after the fourth and eighth readings. The first reading was taken on the staff held on the bench mark of R. L. 820.765.

Rule out a page of a level field book and enter the above readings. Calculate the R.L. of the points and show the usual checks. Find out the gradient of the line joining the first staff position and last staff position if the measured distance between the staff positions is 120.50 metres. **(A.M.I.E. 1983)**

Solution : The staff readings are entered in the level field book and the R. Ls. of the points are calculated -

B. S	I. S.	F. S.	Rise	Fall	Reduced Level	Stn.	Distance	Remarks
0.895					820.765	1	0	R. L. of Stn. 1
	1.645			0.750	820.015	2		= 820.765
	2.895			1.250	818.765	3		
0.955		3.015		0.120	818.645	4		
	0.695		0.260		818.905	5		
	0.585		0.110		819.015	6		
1.535		0.250	0.335		819.350	7		Total distance
	0.955		0.580		819.930	8		between Stn.
		2.135		1.180	818.750	9	120.50	1 and 9.
3.385		5.400	1.285	3.300	820.765			
		3.385		1.285	818.750			
		2.015		2.015	2.015			Check

The amount of vertical fall from station 1 to station 9 = 2.015 metres in a distance of 120.50 metres.

Hence the **Gradient of the line** joining the first staff position and the last staff position

$$= 2.015 \text{ in } 120.50 = 1 \text{ in } \frac{120.50}{2.015} = \textbf{1 in 59.801.}$$

Example 19. The following staff readings were recorded during the process of levelling :

Station	Back Sight	Inter sight	Fore Sight
A	1.285		
B		1.670	
C	2.325		1.955
D		2.105	
E		1.905	
F			1.255

Enter the above readings in a Page of a level-book and calculate the reduced levels of the points by the height of Instrument method if the reduced level of starting point is 200.205m. Also apply the usual check.

Solution : In the height of instrument method of reducing levels, add the back sight 1.285 to the R.L of this point A to get the height of collimation. From this subract the readings 1.670, 1.955 to get the respective R. Ls of the points B and C. To the R.L of the point C thus obtained again add the back sight reading 2.325 to get the height of Collimattion for the second set up of the instrument. Substract 2.105, 1.905, and 1.255 from the new height of Collimation to get the respective R.Ls of points D, E and F.

The calculation work is shown on level book page :

Stn.	Dist.	Bearing	Readings			Height of instrument	Reduced Level	Remarks
			B. S.	I. S.	F. S.			
A			1.285			201.490	200.205	Bench Mark
B				1.670			199.820	
C			2.325		1.955	201.860	199.535	Change Point
D				2.105			199.755	
E				1.905			199.955	
F					1.255		200.605	
		Total	3.610		3.210			

Arithemtical Check :

$$\Sigma \text{B.S} - \Sigma \text{F.S} = 3.610 - 3.210 = 0.400$$
$$\text{Last R.L} - \text{First R.L} = 200.605 - 200.205 = 0.400$$

Hence entries and calculations are correct and the point F is at rise than point A.

Example 20. In a preliminary survey, a line of levels was run from a bench mark of R.L. 454.650 and the following readings were obtained :

Back sights : 2.545, 3.865, 2.670, 1.125
Fore sights : 1.365, 2.945, 1.885

From the last position of the instrument, five pegs at 20 metre interval are to be set on a uniformly rising gradient of 1 in 40. The first peg is to have an R.L. of 455.110. Work out the staff readings required for setting the tops of the pegs on the given gradient and enter them in a level-book page. Work out the R. Ls by collimation method.

Solution : Find the H. I. at the last set up of the instrument from the given back sights and fore sights. Knowing H. I. and the R. L. of the first peg, find the staff reading required for setting top of the first peg. Since the peg interval is 20m and the gradient is rising 1 in 40, therefore R. L. of each successive peg will be more than the preceding by 20/40 = 0.50m and consequently the reading will be less by the same amount. Readings for 1st to 4th pegs will be entered in the column of intermediate sights and the last reading on peg 5 in the fore sight column.

Height of instrument at the last set up = 458.660
Subtract R. L of 1st peg 455.110
Staff reading on the 1st peg 3.550
Subtract rise between successive pegs 0.500
Staff reading on the 2nd peg = 3.050
Subtract rise between successive pegs 0.500
Staff reading on the 3rd peg = 2.550
and so on.

The results are tabulated as under -

Station	Distance In metres	Readings			Height of Instrument	R. L	Remarks
		B. S.	I. S.	F. S			
A		2.545			457.195	454.650	B.M.
B		3.865		1.305	459.695	455.830	C. P
C		2.670		2.945	459.420	456.750	C. P.
D		1.125		1.885	458.660	457.535	C. P.
	0		3.550			455.110	Peg No. 1
	20		3.050			455.610	Peg No. 2
	40		2.550			456.110	Peg No. 3
	60		2.050			456.610	Peg N0. 4
	80			1.550		457.110	Peg No. 5
	Total	10.205		7.745			

Arithmetical Check :

Σ B. S. $-$ Σ F. S $= 10.205 - 7.745 = 2.460$
Last R. L. $-$ 1st R. L. $= 457.110 - 454.650 = 2.460$

Hence entries and calculations are correct.

Example 21. The following readings were taken in running a line of levels from a B.M. having R. L. = 100.00m.

Back sight : 1.290 (on a B. M.), 3.115, 0.235, – 1.125 (on the bottom of the lintel), 2.775.
Fore sight : 0.025, 0.975, – 1.095 (on the bottom of lintel), 3.565 (on a point E).

From the position of the instrument 4 pegs are to be set up at an interval of 30 metres such that the top of the pegs are at a rising gradient of 1 in 100. Work out the staff readings required to fix the pegs on the given gradient. Enter the result on a page of level-book.

Solution : For fixing the pegs at 30m interval and rising at 1 in 100, the difference of R. L for each 30m $= \frac{30}{100} = 0.300$m. Therefore, after calculating the R. L. of the point E in the table, the Rise column against the pegs is filled up first by entering the figure 0.300m and respective R.Ls are also calculated and entered in the table. As the pegs are rising, their staff readings must be lower than the preceding one by 0.300m.

The calculation work is shown in a table -

Stn.	Dist.m	B. S	I. S.	F. S.	Rise	Fall	R. L	Remarks
A		1.290					100.00	B.M.
B		3.115		0.025	1.265		101.265	C.P
C		0.235		0.975	2.140		103.405	C.P
D		-1.125		-1.095	1.330		104.735	C.P. on bottom of lintel
E	0	2.755		3.565		4.690	100.045	C.P.
1	30		2.455		0.330		100.345	Peg No.1
2	60		2.155		0.300		100.645	Peg No. 2
3	90		1.855		0.300		100.945	Peg No. 3
4	120			1.555	0.300		101.245	Peg No. 4
	Total	6.270		5.025	5.935	4.690		

Arithemtical Check :

B. S. – F. S. = 6.270 – 5.025 = 1.245
Rise – Fall = 5.935 – 4.690 = 1.245
Last R. L. – 1st R. L. = 101.245 – 100.00 = 1.245.

Example 22. During the construction work of a building the value of the Bench Mark having R. L. 250.250m was to be shifted near the building on a pillar constructed. The bench mark was below the bottom of a balcony. The following observations were made :

Reading on an inverted staff kept at B. M = 1.325m
Reading on the top of the pillar constructed = 1.015m
Back sight on the pillar constructed = 1.120m
Reading of inverted staff on the bottom of cornice = 2.860m

Enter the readings in a level-book page and calculate the R. L. of cornice.

Solution : Since the first reading is – ve and the next reading is + ve, there will be a fall of – 1.325 – 1.015 = –2.340m.

The last reading is also an inverted staff reading, therefore the station is at a rise of 1.120 + 2.860 = 3.980m.

Stn.	Readings			Rise	Fall	R. L	Remarks
	B. S	I. S.	F. S.				
1	– 1.325					250.250	B.M. at the bottom of a Balcony.
2	1.120		1.015		2.340	247.910	Top of pillar constructed.
3			– 2.860	3.980		251.890	Bottom of cornice of building.
Total	– 0.205 –(–1.845)		–1.845	–3.980 2.340	2.340	251.890 –255.250	
	1.640			1.640		1.640	

Example 23. The following are the notes of a levelling along a mine roadway. The figures relate to inverted staff readings taken for roof heights. Work out the reduced level of roof at each station and apply conventional arithemtical checks on results. Thereafter calculate thickness of roof ripping required at each station to give an even grade of roof. Floor is uniformly rising between station 1 and 7 and height available between roof and floor at station 1 and 7 required all through is 2.4m. All measurements are in metres.

B. S	I. S.	F. S.	Horizontal Distane	Reduced Level		Remarks
				Floor	Roof	
0.32			0	0.00	2.40	Station 1
	0.73		15			Station 2
	1.04		30			Station 3
0.10		0.57	45			Station 4
	0.87		60			Station 5
	0.46		75			Station 6
		1.65	90			Station 7

(Survey Comp. 1973)

Solution : All the staff readings are inverted, therefore, they are to be booked with "Minus" sign or inside brackets. The calculated result is shown in the table :

Stn.	B. S.	I. S	F. S	Rise	Fall	Reduced Level		Dist. mts	Proposed Roof R.L.	Roof Ripping.
						Floor	Roof			
1	– 0.32					0.00	2.40	0	2.40	
2		– 0.73		0.41		0.30	2.81	15	2.70	
3		– 1.04		0.31		0.60	3.12	30	3.00	
4	– 0.10		– 0.57		0.47	0.90	2.65	45	3.30	0.65
5		– 0.87		0.77		1.20	3.42	60	3.60	0.18
6		– 0.46			0.41	1.50	3.01	75	3.90	0.89
7			– 1.65	1.19		1.80	4.20	90	4.20	
	0.42		2.22	2.68	0.88		4.20			
			0.42	0.88			2.40			
			1.80	1.80			1.80			

The floor is uniformly rising from station 1 to station 7 for a distance of 90 metres and the total vertical rise for this distance is 1.80 metres. The distance between the successive stations is 15 metres, hence, the increase in R. L. $= \frac{1.80 \times 15}{90} = 0.30$ metres for successive floor stations . The R. L. of roof stations have been calculated by adding 2.4 with respective R.Ls of the floor stations.

Emample 24. The following details were obtained in levelling along the centre line of a proposed surface road :

B. S	I. S.	F. S.	Station	Distance in metre
1.81			A	0
	0.82		B	37
0.92		0.32	C	63
	0.73		D	113
0.87		1.15	E	152
1.43		2.18	F	162
		0.74	G	201

Calculate the gradient of line A to G. and the cutting and filling required for uniformly graded road from A to G. **(Survey Comp. 1981)**

Solution : Calculations for Reduced Levels are given in the table -

Stn.	B. S	I. S.	F. S.	Rise	Fall	R. L	Distance metres	Remarks
A	1.81					10.00	0	Assume R.L of A = 10m
B		0.82		0.99		10.99	37	
C	0.92		0.32	0.50		11.49	63	
D		0.73		0.19		11.68	113	
E	0.87		1.15		0.42	11.26	152	
F	1.43		2.18		1.31	9.95	162	
G			0.74	0.69		10.64	201	
	5.03		4.39	2.37	1.73	10.64		
	4.39			1.73		10.00		
	0.64			0.64		0.64		Checked

Difference in level between A and G = 10.64 – 10 = 0.64m

Gradient of line A to G = 0.64m in 201m or 1 in 314 rising.

Calculation for Cutting or Filling :

Stn.	Distance	Proposed R. L.	Actual R.L.	Cutting	Filling
A	00	10.000	10.00		
B	37	$10 + {}^{37}/_{314} = 10.118$	10.99	0.872	
C	63	$10 + {}^{53}/_{314} = 10.200$	11.49	1.290	
D	113	$10 + {}^{113}/_{314} = 10.360$	11.68	1.320	
E	152	$10 + {}^{152}/_{314} = 10.448$	11.26	0.776	
F	162	$10 + {}^{162}/_{314} = 10.516$	9.95		0.566
G	201	$10 + {}^{201}/_{314} = 10.640$	10.64	Nil	Nil

Example 25. The following are the notes of a levelling along a railway track which has been affected by a subsidence. Points A and B are outside the affected area and the gradient between them was originally uniform.

B. S.	I. S.	F. S.	Distance	Remarks
1.26			0	A
	0.89		30	
1.25		0.53	60	
	0.97		90	All the
	0.62		120	measurements
1.13		0.18	150	in metres
1.20		0.64	180	
	0.79		210	
		0.40	240	B

Calculate the depth of filling at 15m intervals to restore the track to the original gradient.
(L. Min. E. 1977)

Solution : The calculated result is shown in the table -

B. S	I. S.	F. S.	Rise	Fall	R. L	Dist.m	Remarks
1.26					100.00	0	A B. M. 100
	0.89		0.37		100.37	30	
1.25		0.53	0.36		100.73	60	
	0.97		0.28		101.01	90	
	0.62		0.35		101.36	120	
1.13		0.18	0.44		101.80	150	
1.20		0.64	0.49		102.29	180	
	0.79		0.41		102.70	210	
		0.40	0.39		103.09	240	B
4.84		1.75	3.09	0.00	103.09		
1.75			0.00		100.00		
3.09			3.09		3.09		Checked

Difference in R. L. Between A and B is 3.09m in 240m.

The railway track is rising uniformly from A towards B and increase in rise for each 15 m

$= \frac{3.09 \times 15}{240} = 0.193$m. The proposed R.Ls. at 15m interval is calculated accordingly and the Depth of Filling to restore the track to the original gradient is calculated deducting the present R.Ls. from the proposed R. Ls. at respective points.

Point	0	15	30	45	60	75	90	105	120	135	150	165	180	195	210	225	240
Existing Reduced Level	100.00	100.185	100.370	100.550	100.730	100.870	101.010	101.185	101.360	101.580	101.800	102.045	102.290	102.495	102.700	102.895	103.090
Increase in Reduced Level at 15m interval	00	0.193	0.336	0.579	0.772	0.966	1.160	1.354	1.547	1.740	1.933	2.126	2.319	2.512	2.705	2.897	3.090

Point	0	15	30	45	60	75	90	105	120	135	150	165	180	195	210	225	240
Proposed Reduced Level	100.00	100.193	100.386	100.579	100.772	100.966	101.160	101.354	101.547	101.740	101.933	102.126	102.319	102.512	102.706	102.897	103.090
Depth of Filling	00	0.008	0.016	0.029	0.042	0.096	0.150	0.169	0.187	0.160	0.133	0.081	0.029	0.017	0.005	0.002	00

Example 26. The following is a page from a levelling field book. Fill in the missing Data.

St.	B. S	I. S.	F. S.	Rise	Fall	Reduced level	Distance
1	0.50						00m
2	1.21				0.99		20m
3	2.94		0.42				40m
4				1.53			70m
5		1.63			0.22	250.00	80m
6	2.21			0.37			100m
7		0.84					130m
8					1.63		150m

(L. Min. E. 1980)

Solution : The F. S. of station 2 is missing, but the fall is given. Since station 1 is higher, therefore the fore sight of 2 = 0.50 + 0.99 = 1.49.

The I.S. of station 4 is missing, but the rise is given. Being at rise, the I.S. of station 4 is less than the reading at station 3. The reading of station 4 = 2.94 – 1.53 = 1.41.

Now complete columns of rise and fall after calculating them. From the known rises and falls the reduced levels of other points can be calculated. While calculating the RLs. of station 6th and 7th, the rise is added to the R.L. of the preceding point and for station 8, the fall is subtracted from the R. L. of station 7. For calculating R.Ls from station 4 upwards to station 1, the rise is to be substracted and fall is to be added to the R.L. of the preceding point i.e. the signs of rise and fall are to be reversed while calculating upwards .

The result is shown in the table -

St.	B. S	I. S.	F. S.	Rise	Fall	Reduced Level	Distance	Remarks
1	0.50					248.90	00m	
2	1.20		1.49		0.99	247.91	20m	
3	2.94		0.42	0.78		248.69	40m	
4		1.41		1.53		250.22	70m	
5		1.63			0.22	250.00	80m	R.L. given
6	2.21		1.26	0.37		250.37	100m	
7		0.84		1.37		251.74	130m	
8			2.47		1.63	250.11	150m	
	6.85		5.64	4.05	2.84	250.11		
	5.64			2.84		248.90		
	1.21			1.21		1.21		Checked

Example 27. Following is the page of a level book. Some of the readings are missing which are marked by cross mark (**X**). Calculate the missing readings and apply the check :

Back Sight	Inter sight	Fore sight	Rise	Fall	Reduced level
x					150.000
	2.455			0.825	x
	2.400		0.055		x
2.695		x		x	148.070
x		2.050	0.645		148.715
	2.500		1.070		149.785
	2.895			x	149.390
	x			0.125	x
		2.670	0.350		149.615

(A.M.I.E. 1976)

Solution : Refer to table where the result of this example is tabulated.

The R.L. of point A = 150.00m, the fall being 0.825m at B, therefore R.L. of B = 150.00 – 0.825 = 149.175m. The rise at C is given as 0.055 m at B, therefore R.L of C = 149.175 + 0.055 = 149.230m. The reduced level of point D = 148.070 (given), Since 1.160 is a fall, the fore sight reading should be more.

The fore sight = 2.400 + 1.160 = 3.560m

The inter sight at point B = 2.455 and fall is 0.825.

The inter reading at F and the rise = 2.560m = 1.070 (given)

Therefore, B.S. of A = 2.455 - 0.825 = 1.630

Being rise the back sight at point E will be more than the inter sight at F.

Back sight at E = 2.500 + 1.070 = 3.570m

The inter sights of point F and G are 2.500, 2.895, as the reading following point F is more, hence there is a fall 2.895 - 2.500 = 0.395 (fall).

The R.L. of point G = 149.390 and fall at H = 0.125

R.L. of point H = 149.390 - 0.125 = 149.265

The rise at point J = 0.350

Inter sight of point H = 2.670 + 0.350 = 3.020m

The results are tabulated in the following table -

Station		Readings			Rise	Fall	Reduced level	Remarks
		B. S.	I. S.	F. S.				
A		1.630					150.000	B. M.
B			2.455			0.825	149.175	
C			2.400		0.055		149.230	
D		2.695		3.560		1.160	148.070	C. P.
E		3.570		2.050	0.645		148.715	C. P.
F			2.500		1.070		149.785	
G			2.895			0.395	149.390	
H			3.020			0.125	149.265	
J				2.670	0.350		149.615	Last point
	Total	7.895		8.280	2.120	2.505		

Arithemtical Check :

Σ B. S $-$ Σ F. S $=$ Σ Rise $-$ Σ Fall $=$ last R. L. $-$ 1st R. L.

7.895 $-$ 8.280 $=$ 2.120 $-$ 2.505 $=$ 149.615 $-$ 150.00

$-$ 0.385 $=$ $-$ 0.385 $=$ $-$ 0.385 (fall)

$=$ $-$ 0.385 (fall)

Example 28. The following is the page of a level field book. Fill in the missing reading and calculate the reduced level of all the points . Apply usual check.

Stn.	B. S.	I. S.	F. S.	Rise	Fall	Reduced Level	Remarks
1	3.250						Starting Point
2	1.755		?		0.750		Change point
3		1.950					
4	?		1.920				Change point
5		2.340		1.500			
6		?		1.000			
7	1.850		2.185			250.000	B. M.
8		1.575					
9		?					
10	?		1.895		1.650		Change point
11			1.350	0.750			Last point

(A.M.I.E. 1974)

Solution : (i) The F. S. of station (2) is missing, but fall is given. Since station (1) is higher, therefore the fore sight (2) = 3.250 + 0.750 = 4.000m.

(ii) The inter sight of station (5) is given and from the known rise, the back sight of station (4) can be found. As (5) is at rise, the back sight of (4) will be more hence back sight = 2.340 + 1.500 = 3.840m.

(iii) I. S. of station (6) is missing, from the known rise its reading can be found. Being at rise, the reading will be less than station (5) = 2.340 – 1.000 = 1.340m

(iv) The F. S station (10) is given from the known fall 1.650, the I. S. of station (9) can be found. As (10) is at fall the reading of (9) will be less = 1.895 – 1,650 = 0.245m.

(v) The reading of station (10) is missing, by known rise of station (11), the back sight reading of (10) can be found. The reading will be more than station (11) as (11) is at rise = 1.350 + 0.750 = 2.100m.

(vi) Now complete the column of rise and fall after calculating them. From the known rises and falls the reduced levels of other points can be calculated. While calculating the R.Ls of stations from 8th to 11th, the rise is added to the R.Ls of preceding point and fall is subtracted. From calculating R.L. from station 6th to 1st, the rise is to be subtracted and fall is to be added to the R.Ls of preceding point when going upward i.e. the signs of rise and fall are to be reversed.

The result is shown in the table -

Station	Dist in M	Readings			Rise	Fall	Reduced Level	Remarks
		B. S.	I. S.	F. S.				
1		3.250					249.260	Starting point
2		1.755		4.000		0.750	248.510	C. P.
3			1.950			0.195	248.315	
4		3.840		1.920	0.030		248.345	C. P.
5			2.340		1.500		249.845	
6			1.340		1.000		250.845	
7		1.850		2.185		0.845	250.000	Bench Mark
8			1.575		0.275		250.275	
9			0.245		1.330		251.605	
10		2.100		1.895		1.650	249.955	C. P.
11				1.350	0.750		250.705	Last point
	Total	12.795		11.350	4.885	3.440		

Arithemtical Check :

Σ B. S $-$ Σ F. S $=$ Σ Rise $-$ Σ Fall $=$ Last R. L. $-$ 1st R. L.

12.795 $-$ 11.350 $=$ 4.885 $-$ 3.440 $=$ 250.705 $-$ 249.260

1.445 $=$ 1.445 $=$ 1.445 (+ ve)

Example 29. The following observations were taken by means of a levelling instrument.

A borehole is to be driven from point C for sand stowing purpose upto the roof of the seam. find out the depth of the borehole assuming the seam to be uniform gradient.

Stn.	B. S.	I. S.	F. S.	Rise ●	Fall ●	R. L ●	Dist. m	Remarks
A	4.25					50	0	50m A. D
1		5.50			1.25	48.75	50	
B	6.50		6.25		0.75	48.00	100	Out crop
2		7.40			0.90	47.10	150	
3		8.15			0.75	46.35	200	
C	5.20		10.00		1.85	44.50	250	Borehole point
4		7.00			1.80	42.70	300	
5		5.05		1.95		44.65	350	
6		4.60		0.45		45.10	400	
D			6.70		2.10	43.00	450	Staple pit depth 30m in roof of the seam

(● Columns have been substituted after calculation) **(Survey Comp. 1977)**

Solution : Distance from outcrop of seam to staple pit point = BD = 450 – 100 = 350m.

Gradient of seam = 30 + (48 – 43) = 35m in 350m = 1 in 10.

Distance from B to C (Borehole point) = 250 – 100 = 150m.

Difference of reduced levels along roof seam from B to C = $^{150}/_{10}$ = 15m,

Difference of sorface levels between B and C = 48.00 – 44.50 = 3.50;

Depth of borehole at C upto the roof of the seam = 15m – 3.50 = **11.50m.**

EFFECT OF CURVATURE AND REFRACTION

CURVATURE : The earth being a sphere, the term curvature refers to the curve of its surface. A strictly level line is one in which all the points are equidistant from the centre of the earth. In case of long sights, the horizontal line is not a level line. The level line falls away from the horizontal line and the vertical distance between the horizontal line and the level line represents the effect of the curvature of the earth.

Effect of Curvature : The curvature of earth causes an object on its surface to appear lower

than it really is with reference to the point of observation and therefore, the staff-reading appear greater than it really is. The correction is subtractive.

Correction for Curvature : The correction for curvature may be derived as follows -

Level line through the instrument axis = AC
Horizontal line (Line of collimation) = AB
Centre of earth = O
Radius of the earth = R (km)
Diameter of the earth = 2R = 12742 (km)
Distance from the instrument station to staff = D (km)
Correction for curvature = BC

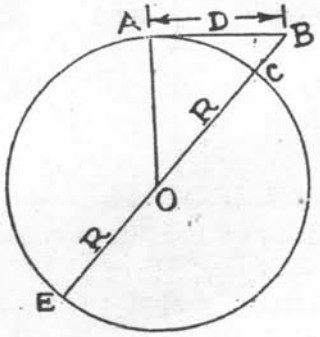
Thus in figure, $BO^2 = AB^2 + AO^2$ .....(Euc. 1.47)

But $BO = BC + CO$ $\therefore (BC + CO)^2 = AB^2 + AO^2$

Again $CO = AO$ $\therefore (BC + AO)^2 = AB^2 + AO^2$

or, $BC^2 + 2BC.\ AO + AO^2 = AB^2 + AO^2$

or, $BC^2 + 2BC.\ AO = AB^2$



But, BC must always be relatively very small compared with the radius of the earth. The square of BC will be still smaller and it may be neglected in calculation, and the equation becomes -

$$BC = \frac{AB^2}{2AO} = \frac{D^2}{2R} \text{ in Km} = \frac{D^2}{12742} \text{ in km.}$$

or, the correction for curvature = BC (in metres) $= \frac{D^2 \times 1000}{12742} = 0.0785\ D^2$

The correction is subtractive from the observed staff reading.

REFRACTION : Rays of light passing through layers of air of different densities are refracted or bent down. The effect of refraction in making objects appear higher than they really are (and thus the staff reading too small) can be remembered by considering what happens when the sun is on the horizon. At the moment the sun has just passed below the horizon, it will be seen just above the horizon. In the early morning or late in the evening, when the air is warmer than the ground or vice versa, refraction tends to be greater close to the ground than some distance above it. The ray of light from the staff to the instrument is not straight, but follows a curved path. This curved path - concave towards the earth, is an arc of a circle of radius equal to seven times that of the earth under normal conditions of the atmosphere. The effect of refraction is not constant, but varies with atmospheric conditions.

Effect of Refraction : As refraction causes all objects to appear higher than they really are, staff-readings appear less than they really are although, in fact, it varies appreciably at different times of the day and in different climates.

Correction for Refraction : The corection for refraction is on an average, one-seventh that of curvature and acting as it does in the opposite direction. Therefore, air refraction neutralises to some extent the amount of error to the earth's cuvature.

The correction for refraction $= {}^{1}/_{7}\ (0.0785D^2) = 0.0112D^2$ in metres.

The correction is additive to the observed staff-reading.

COMBINED CORRECTION : In precise levelling work or when the sights are long, the effects of curvature and refraction have to be taken into account. In ordinary levelling corrections for curvature and refraction are very limited. It is about 100 metres with ordinary levels and with levels of best type about 200 metres. The combined error for a sight ranging 200m is 0.003m. The errors, due to the instrument not being in perfect order, are much greater than these casued by curvature and refraction. The combined error can, however, be eliminated (i) by equalising the distance of backsights and foresights, and (ii) by the process of reciprocal levelling.

Since the combined effect of curvature and refraction is to increase the staff readings, the combined correction is subtractive.

The combined correction $= 0.0785D^2 - 0.0112D^2 = 0.0673D^2$ metres.

There are two ways of applying the correction : (i) each staff-reading is decreased by the amount of combined correction ($0.0673D^2$), and (ii) the reduced level of the point is obtained in the usual way, and is then increased by the amount $0.0673D^2$ metres.

Example 30. (a) Find the correction for curvature for a distance 800m.

(b) Find the correction for refraction for a length of sight 600m.

(c) Find the combined correction for curvature and refraction for a length of sight 2.5 km.

Solution : (a) Correction for curvature $= 0.0785D^2$ metres.
Given, Distance, D = 800 metres = 0.8 km.
Hence, *Correction* $= 0.0735\ (.8)^2 = 0.05$m.

(b) Correction for refraction $= 0.0112D^2$ metres.
Given, D = 600m = 0.6 km.
Hence, *Correction* $= 0.0112 \times (0.6)^2 =$ **0.004 m.**

(c) Combined correction $= 0.0673D^2$ metres $= 0.0673 \times (2.5)^2$ m $= 0.4206$m.

Example 31. A level is set up at C on a line AB, 84m from A and 700m from B. The back sight on A is 1.025m and the fore sight B is 3.305m. Find the true difference of level between the two stations.

Solution : The length of sight CA is small, hence no correction is required, but the length of sight CB is more (700m = 0.7km), therefore, correction for curvature and refraction has to be applied to the staff reading taken at B.

Combined correction for 700m	$= 0.0673 \times (0.7)^2$	= 0.033m.
Hence correct staff reading at B	$= 3.305 - 0.033$	= 3.272m.
True difference of level between A and B	$= 3.272 - 1.025$	**= 3.247m (fall)**

CLASSIFICATION OF LEVELLING

Levelling may be classified as follows -

1. Simple levelling.
2. Differential or Compound levelling.
3. Flying levelling.
4. Check levelling.
5. Recriprocal levelling.
6. Longitudinal levelling.
7. Cross levelling or cross-sectioning.
8. Precise levelling.

1. SIMPLE LEVELLING : This term is applied when the difference in level between two points can be obtained from one setting of the instrument. If it is required to find the difference in level between points A and B, the instrument is set up at a convenient position C, preferably midway between the points. The instrument is levelled until the bubble of the main spirit level remains central throughout a complete revolution of the instrument. In the event, the line of collimation will be truly horizontal. A back sight is taken to a levelling staff held at A and, thereafter, foresiget to the staff at B. If we suppose that the staff reading at A is 1.320 and that at B 2.020, the difference in level between the two points equals 2.020 – 1.320 = 0.700 metre, falling from A to B . If the altitude of A relative to some datum (reduced level), is known say 102.560, the level of the line of collimation is 102.560 + 1.320 m 103.880 m. The reduced level of B is found by subtracting the fore-sight reading, 2.020, from the level of the line of collimation (103.880 – 2.020), giving a value of 101.860. If the reduced level of point B is subtracted from that of point A (102.560 – 101.860) the difference in level is found (0.700) as before.

2. COMPOUND LEVELLING : When the difference in height between the two staff points is greater than the height of the staff, or the distance between the points is excessive, then the levelling is carried out in sections and the operation is called compound levelling. In order to find out the difference in level between A and B, it may be necessary to set up the instument at C and at E, midway between A and D, and D and B respectively. Let us suppose that the staff readings are as follows :

Instrument set at C		*Instrument set at E*	
Back sight to	A..... 2.505m	Backsight to	D..... 0.475m
Foresight to	D..... 1.080m	Foresight to	B..... 3.225m

Difference in level between A and D = 2.505 – 1.080 = 1.425m point D being at higher level than A.

Difference in level between D and B = 3.225 – 0.475 = 2.750m and this is obviously a fall from D to B.

Hence, the difference in level between A and B = 2.750 – 1.425 = 1.325 metres (fall).

From the above, it is evident, that when the foresight exceeds the preceding backsight,

there is a fall between the two staff stations. This is the fundamental rule of geometrical levelling and should be memorised. The above observations may be recorded in the following manner -

Backsight	Foresight	Rise	Fall	Reduced level	Distance	Remarks
2.505				100.000		A
0.475	1.080	1.425		101.425		Staff station D
	3.225		2.750	98.675		Staff station B
2.980	4.305	1.425	2.750	100.000		
	2.980		1.425	98.675		
Checks	1.325		1.325	1.325		Net fall

In the above table, it has been assumed that the reduced level of A is 100.00m above datum.

3. FLY LEVELLING : In this method, the relative levels of two and more isolated points are determined and no intermediate sights and no distances between staff stations are required. A line of levels run with more rapidity with fly levelling. An example of such a levelling would be from a bench mark, known to be stable, to the top of a mine shaft in order to establish its correct height above datum.

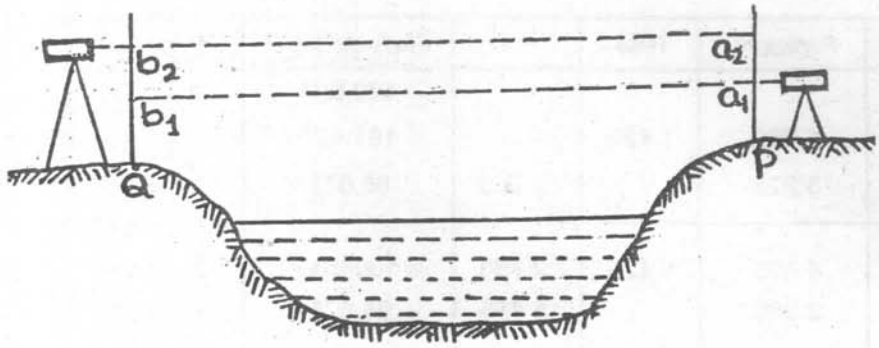
4. CHECK LEVELLING : It is conducted for the purpose of checking a series of levels, which have already been fixed with respect to some bench marks. In this method only back sights and fore sights are taken. There is no need of intermediate sights. Great care has to be exercised for selecting the change points and for taking readings on the change points because the accuracy of levelling depends upon these points.

5. RECIPROCAL LEVELLING : In ordinary levelling, where the sights are comparatively short the effect of refraction is usually ignored and the equalisation of backsights and foresights will eliminate curvature and refraction errors. If the route of levelling has to cross a river it may be impossible to equalise backsights and foresights. In such a case, the errors of curvature and refraction can only be eliminated by adopting the procedure known as *reciprocal levelling.*

Procedure : The procedure, which is a method of reversion, is used in making adjustments of levels and transits. Let the difference of level between two points P and Q on opposits banks of a river is to be found out. In such case it is not possible to set up the level midway between the points.

The instrument is set up very near to P, its height a_1 is measured with a staff at P and a reading b_1 is taken to a staff held at Q.

The instrment is set up very near Q and again two readings from the staff held at P and Q are noted. Let these are a_2 and b_2 respectively.



Reciprocal Levelling

If, d = true difference of level between P and Q.
e = total error (curvature + refraction + collimation)

Then, correct reading on Q in the first setting $= b_1 - e$
and, correct reading on P in second setting $= a_1 - e$

From the first setting of the level, we get

True fall from P to Q = d $= (b_1 - e) - a_1$
$= (b_1 - a_1) - e$

Again, from the second setting of the level, we get

True fall from P to Q $= d = b_2 - (a_2 - e)$
$= (b_2 - a_2) + e$

Avergaing the two results we get

$$d = \frac{(b_1 - a_1) - e + (b_2 - a_2) + e}{2} = \frac{(b_1 - a_1) + (b_2 - a_2)}{2}$$

Advantages : (1) It may be used when it is not possible to set up the level midway between the two points in case of river, valley, etc.
(2) The errors due to curvature and refraction may be entirely eliminated.
(3) Error due to maladjusted line of collimation may be eliminated.

It will be seen that, providing the observations are made simultaneously, errors due to curvature and refraction are eliminated. Since this necessitates the use of two instruments, however, which may both have collimation error, a second set of observations must be taken with the instrument interchanged. The mean of the two results should then be free from curvature, atmospheric refraction and collimation error.

Example 32. In levelling between two points P and Q on the opposite banks of a stream, the level was set up close to P and the staff readings on P and Q were 1.425 and 2.045m respec-

tively. The level was then moved and set up near Q. The readings on the staff at P and Q were 0.740 and 1.320m. Find the true difference of level between P and Q. Also determine the reduced level of peg Q if that of P was 200.050m.

Solution : *Level at P :* Staff reading on P = 1.425m
Staff reading on Q = 2.045m
Apparent difference of level between P and Q
= 2.045 – 1.425 = 0.620m (fall from P)
Level at Q : Staff reading on P = 0.740m
Staff reading on Q = 1.320m
Apparent difference of level between P and Q
=1.320 – 0.740 = 0.580 (fall from P)

True difference of level between P and Q $= \frac{0.620 + 0.580}{2}$ **= 0.600m.**

The R. L. of P = 200.050m
R. L. of Q = 200.050 – 0.600 = **199.450m.**

6. LONGITUDINAL SECTIONING OR LEVELLING : The object of this levelling operation is to obtain a record of the undulations of this surface along the centre line of a proposed road, railway, or canal, etc, The outline of the surface thus obtained is called the *longitudinal section* or *profile* so why this levelling is also known as **profile levelling**. From such a section, the surveyor is in a position to study the relationship between the existing ground surface and the proposed formation of the new work in the direction of its length.

The operation involves observing the elevations of a number of points along the centre line and also their distances along it. The line of section may be a single straight line running in the centre or may cosist of series of straight lines changing direction or connected by curves. The levels are taken at uniform intervals of distance along the centre line depending upon the requirements of the work and nature of the ground. Beside these points the staff readings are taken at the fairly significant points where the outline of the ground changes appreciably so that the profile may be obtained as natural as possible.

If the levelling starts from a permanent bench mark and ends on a permanent bench mark the difference between the sums of back and fore sights must agree with the difference of levels between the first and the final bench mark within permissible limit of the closing error. And if the levelling work is not closed on the permanent bench mark, the only way to check it is to take flying levels to the original bench mark and finding the closing error. If the closing error exceeds the permissible value the work must be repeated.

Plotting the Section. In plotting the longitudinal section, a horizontal line is drawn as datum line, and chainages of the staff points are marked along with this line to a convenient scale. At these plotted points, perpendiculars are erected and on each of these lines, the respective levels are set off. The plotted points are then joined by straight lines to obtain the out lines of the ground surface. The horizontal scale used in plotting should be the same as that of the plan but the vertical scale in plotting the levels should be larger so as to mark the ground inequalities more apparent.

The plotted plan shows the features of the original ground surface, the formation levels of new work, the proposed gradient, the depths of cutting and heights of filling, and any other information which may be useful during construction of the work.

Example 33. The following are notes of an underground levelling :

B. S	I. S	F. S	Hor. Distance	Remarks
1.380			0m	A.(3m above an assumed datum)
	0.885		30m	
1.464		0.540	60m	
	0.900		90m	
0.255		0.831	120m	
	1.185		135m	
	1.020		150m	
		0.828	180m	

The levels are taken along the floor of an underground roadway which it is proved to grade evenly between A and B to increase the height of ripping, where necessary, from the present height of 1.65 metres to 2.25 metres.

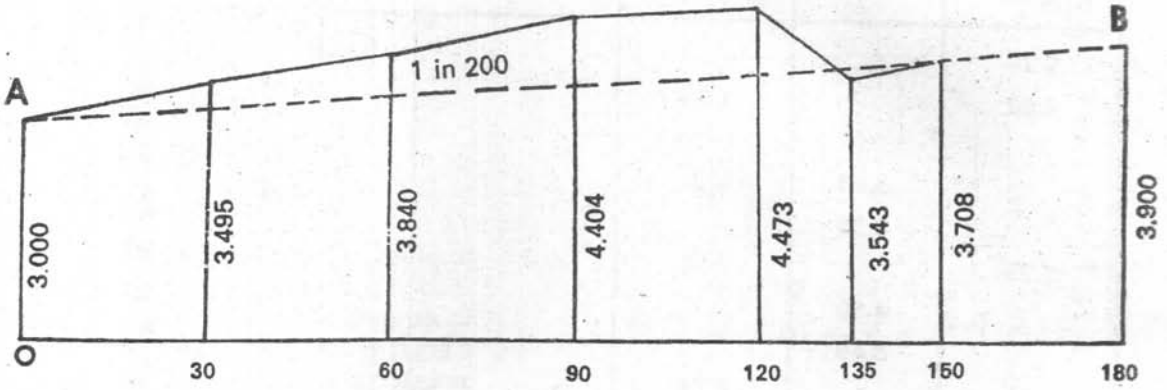
Reduce the levels and plot a section of the roadway showing existing road and the proposed road. The horizontal scale is 1 cm = 12m and the vertical scale is 1cm = 1m. Calculate the cutting or filling of the floor at each point and the roof ripping required.

Solution : The levels are reduced in the field book as follows -

B. S.	I. S.	F. S.	Rise	Fall	R. L.	Distance (metres)	Remarks
1.380			—		3.000	0	A
	0.885		0.495		3.495	30	
1.464		0.540	0.345		3.840	60	
	0.900		0.564		4.404	90	
0.255		0.831	0.069		4.473	120	
	1.185			0.930	3.543	135	
	1.020		0.165		3.708	150	
		0.828	0.192		3.900	180	B
3.099		2.199	1.830	0.930	3.900		
2.199			0.930		3.000		
0.900			0.900		0.900		

Gradient between A and B = (3900 – 3000) m in 180 = 1 in 200

Rise from A to B per 30 m = 0.15m.



Section of the roadway

Station	A	30	60	90	120	135	150	180
Existing floor level	3.000	3.495	3.840	4.404	4.473	3.543	3.708	3.900
New floor Level	3.000	3.150	3.300	3.450	3.600	3.675	3.750	3.900
Floor cut	–	0.345	0.540	0.954	0.873	–		–
Floor fill	–	–	–	–	–	0.132	0.042	-
Existing roof level	4.650	5.145	5.490	6.054	6.123	5.193	5.358	5.550
New roof level	5.250	5.400	5.550	5.700	5.850	5.925	6.000	6.150
Roof rip	0.600	0.255	0.060	Nil	Nil	0.732	0.642	0.600

7. CROSS LEVELLING OR CROSS SECTIONING : Cross-sectioning is done generally at right angles to the centre line and on either side of it for the purpose of knowing the lateral outline of the ground surface. Sometimes oblique cross-sections may also be taken. Cross-sections provide data for estimating the quantities of earth work and for other purposes. The length of the cross-section depends upon the nature of work. The peg on interval on either side may be suitably chosen.

The longitudinal and cross-section may be taken together or separately and preferably at L-section points. To avoid confusion the booking of each cross-section should be done separately. Two more distance columns are required in the level book to reproduce left and right of the centre line.

Example 34. The following observations were taken by means of a dumpy level. Complete the booking by height of instrument method and plot the same on a horizontal scale of 1cm = 100m and a vertical scale of 1cm = 10m.

B. S	I. S.	F. S.	Instrument height	R. L.	Distance in metres
5.39				50.00	0
4.52		1.05			50
	1.92				100
	5.95				150
	8.39				200
1.75		7.50			250
	4.50				300
	8.48				350
		8.74			400

Solution : The calculations are tabulated as follows -

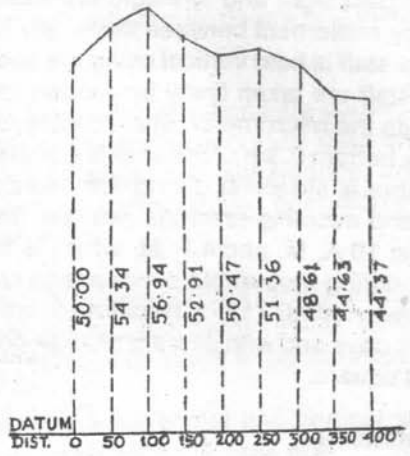
B. S	I. S.	F. S.	Instrument height	R. L.	Distance in metres
5.39			55.39	50.00	0
4.52		1.05	58.86	54.34	50
	1.92			56.94	100
	5.95			52.91	150
	8.39			50.47	200
1.75		7.50	5.311	51.36	250
	4.50			48.61	300
	8.48			44.63	350
		8.74		44.37	400
11.66		17.29		50.00	
		11.66		44.37	
Differences		5.63		5.63	Checked

Plotting : The level section is plotted from the calculated reduced levels taking the datum level as 00.00.

Scale :

Horizontal 1cm = 100m

Virtical 1cm = 10m.



Level Section

8. PRECISE LEVELLING : The term Precise Levelling is intended to convey the idea that a high degree of pricision may be attained in making a levelling of the boundary of a polygon, in levelling from one station to another, in establishing bench marks at widely distributed points, or in recording mining subsidence.

There is no difference in principle between precise and ordinary levelling. In ordinary levellinng, where a standard of accuracy ±s 2cm per kilometre is required, a number of errors at intermediate points can be concealed by the closure. These errors, due to the relative coarseness of their determination, can be compensating and so remain as an inherent part of the survey. In precise levelling all such errors are eliminated by adopting a higher standard of instrument work and a better observation technique. *The standard of accuracy in precise levelling is 1mm per kilometre.*

Instruments Required : (1) A high grade level equipped with tilting screw, stadia wires, coincidence level and opitcal micrometer adjusted before use; (2) Two invar pricision staves reading up to $^1/_{1000}$ m with the optical micrometer, spot bubble on back or attachment of plumb bob, (3) Bolts 30cm long with round heads; (4) Pegs; (5) Shield for the instrument.

Procedure : The area to be surveyed is first examined and a series of bolts are established in line at approximate intervals of 80 to 90m to indicate the staff stations. The instrument is set up midway between two staff stations to equalise backsight and foresight. Both the staves are used so that when the instrument is set at station 1 readings of backsight and foresight are taken on staff A and staff B respectively. Staves are allowed to leap-frog each other, remaining at a station for two readings except the first backsight. At every setting of the instrument except the first setting the foresight is read first. The levelling is closed over the same route but staff B is used as those stations preveously occupied by staff A

Presonnel Required : One observer, one booker, and two staff holder total four persons in a team.

Precautions : (1) Adjustments of the level is carefully tested and the staves are calibrated before use. (2) The instrument is firmly planted at each setting, if the ground is soft the tripod is set on pegs. (3) the instrument is levelled by the foot screws, and the bubble is very

carefully centred by micrometer screw and kept under observation at the time of reading the staff. (4) The length of the sight is limited according to the instrument (generally 40 - 50m), and the backsight and foresight are exactly balanced by stadia readings. (5) To minimise errors by settlement between them. (6) The level is protected from the sun or wind by shield. (7) The staff is held vertical using the spot bubble or plummet for the purpose. (8) Readings on the staff are taken firstly by precise levelling of the telescope and then reading staff fractions with the micrometer. Staff reading is noted first and then the parallel plate mincrometer. (9) The bottom 0.3m of the staff is not used to avoid irregular refraction over the ground. (10) Refraction is steadiest during the mid-day but shimmer is at its maximum. By using short sights and avoiding readings near the floor this is avoided. The levelling work is carried out between 10 A. M. and 4 P. M. which is the best time for observations (11) The levelling is carried during favourable atmospheric conditions of light and temperature. Rainy and windy days are avoided. (12) Check levels are taken by diferent surveyor in opposite direction on different days and with diferent change point. (13) Field observations are adjusted by method of least square.

BOOKING.

Line No. _____ Direction _____ Fore/Back

Date _____ Time _____ Observer _____

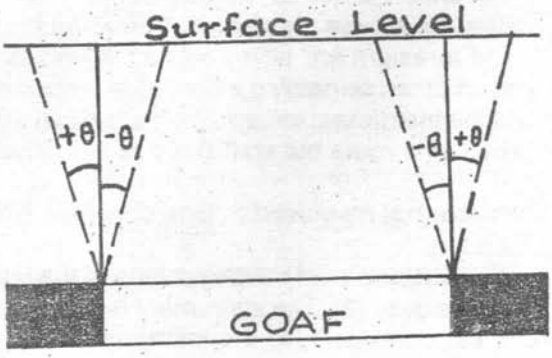
Instrument used _____ Booker _____

Staff No. __A ___ & ____ B

Stn	Staff No.	B. S.	F. S.	Stadia Reading			Remarks
				Top	Bottom	Intercept	

SUBSIDENCE MEASUREMENTS

Subsidence : The term subsidence means the lowering of the surface. When an underground opening is established due to extraction of a coal seam or ore body, the original equilibrium of strata is disturbed with resultant stress concentrations. It causes effects on the surface where particles suffer vertical and horizontal displacement, creating *subsidence ba-*



sin or trough (surface depression) which flattens out to sides until it is level with the existing ground. The area of the surface affected above a goaf depends on the *angle of draw,* which is the angle between a vertical line from the edge of the goaf and a line extended to a point at which subsidence tails out to zero. The angle of draw varies with depth, nature and inclination of the strata, and other grological features.

Aims of Subsidence Measurements : If ground movement observations are carried out in a scientific manner, following every depillaring operation, and the observation data are kept properly maintained, many valuable informations such as amount of subsidence, angle of draw, rate of subsidence, etc. can be made available for planning future operations under similar ground conditions and similar operational parameters.

A correct subsedence record is a mass of percisely observed data which helps to devise methods of forecasting, or precalculating, subsidence and this gives results sufficiently accurate to enable some of the problems and costs of surface damage to be alleviated.

On the basis of subsidence survey carried out for the horizontal and vertical surface movements, the five parameters of subsidence may be ascertained. These are :

1. Vertical subsidence.
2. Differential change in the ground slope.
3. Change in the surface curvature.
4. Horizontal displacement of different surface points.
5. Horizontal strain.

The other factors which may be assessed on the basis of these observations are -

1. Angle of draw.
2. Angle of break.
3. Maximum subsidence.
4. Point of inflexion.
5. Effect of the time on surface movements.
6. Non-effective area of extraction.
7. Critical area of extraction.
8. Effect of depth, thickness and inclination of seam, faults and other geological disturbances, stowing and packing, etc.

In order to maintain records of the amount of subsidence it requires periodic measurements of surface movement which consist of -

(a) Vertical movement, and

(b) Horizontal movement.

The interval of time between successive observations of surface movements varies in each case depending upon the depth of working caving or stowing, etc. In case of rapidly

advancing face (7m per week) intervals should be kept 15 days and for slow advancing this may vary from one month to two months.

MEASUREMENT FOR VERTICAL MOVEMENT

For measuring the vertical movement the following instruments are required :

1. Precise levelling instrument.
2. Levelling staff.
3. Steel band.
4. Theodolite or dial
5. Ranging pole, wooden pegs and or arrangement for concrete pillarings.

The levelling should have the following special features -

(a) A level tube of high sensitivity.

(b) A prism device for giving bubble reading in the field of view of telescope.

(c) A parallal plate type micrometer device for giving the smallest possible staff reading.

Procedure : Test line or the line of observation for getting the informations required should be laid out carefully and the distance between the levelling stations should not preferably exceed 30m. Such stations should be fixed not more than 6m apart near the panel barriers. The test lines should be laid out in the direction of and at right angles to line of extraction. These are marked by wooden pegs or by concrete blocks for an important area. These observation stations should be available for a period even after the completion of depillaring operations or total extraction in the area. Two or three Bench Marks are also selected well outside the area from which mining has been done. Respective heights of these Bench Marks above some datum should remain unaltered.

The area under observation are levelled out to determine the reduced levels of selected reference marks. The levelling instrument is placed in the middle of the back and foresights and levelled properly. The readings of the stations are taken with the help of levelling staff kept at the stations. The staff should be provided with a spirit bubble for checking its verticality during observations. Sights longer than 50m should be avoided to ensure good optical definition of the staff. All levelling observations should be carried out in closed circuit to check the error and if closing error is more than the desired value the operation should be repeated.

The levels must be retaken at regular intervals of a month when the depillaring is in progress; and when the depillaring is completed at least once in one year and at lesser frequencies thereafter.

Personnel Required :

(a) Observer - 1 (b) Recorder - 1 (c) Helper - 2

MEASUREMENT OF HORIZONTAL MOVEMENT

In measuring the horizontal movement the following instruments are required :

(1) Steel band, (2) Spring balance (3) Thermometer, (4) Measuring tripods, (5) Anchor rods, (6) Dumpy level, (7) Levelling staff (8) Plumb bob, field book.

Procedure : The steel band is compared with a standard tape. It is then laid out along the test line or line of observation with the anchor rods at both the ends. A spring balance is provided for the proper tension of the band. The two station points from the line of observation are transferred on tripod head with the help of plumb bobs suspended from measuring tripods. The readings from the steel band corresponding to each station point at a tension of 10 kg are recorded. The readings are repeated by shifting the anchor pins a little on either ways and three readings are noted. The temperature and tension applied at the time of observation are also noted. With the help of Dumpy level the difference of height between the tripod heads are determined.

The length between every two consecutive stations are calculated after necessary correction for temperature, tension, slope, sag, etc. The displacement of surface points which causes closely situated two points to come either near to each other or to move away from each other resulting shortening (due to compressive strains) or lengthening (due to tensile strains) at the surface may thus be found out.

BOOKING

Seam..... Thickness..... Depth.....

First reading on.....

Date	Readings at peg nos.								Remarks
	1	2	3	4	5	6	7	8	

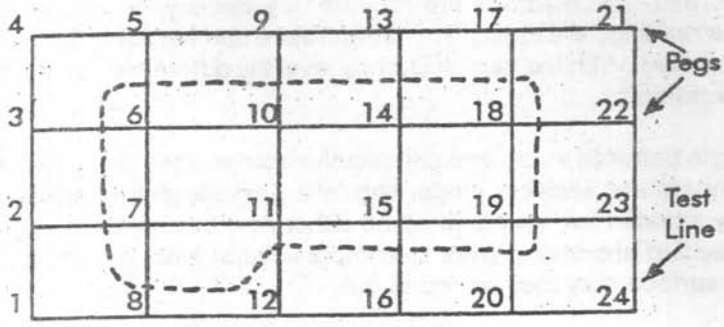
Personnel Required

(a) Observer - 2 (b) Recorder - 1 (c) Helper - 2

Plans and Sections

Separate subsidence plans and sections should be maintained showing -

(a) Observation points in the test line.
(b) The depillared area and/or area under extraction.
(c) Geological distrubances, if any.
(d) Extent of surface subsidence
(e) The method of extraction.
(f) A Section of strata overlying the seam under extraction.
(g) The angle of draw.
(h) The amount of subsidence from time to time.



Subsidence Plan

The plans should be prepared at the same scale at which the statutory plans are maintained. Scale for vertical section should be 10cm to 1m showing the total subsidence with the surface profile and the relative position of the seam (s) and working plotted to a natural scale.

SHAFT DEPTH MEASUREMENT

Object : The object of shaft depth measurement may be mainly as follows -

1. For correlation of surface and underground workings.
2. For establishing the level of a point at bottom of a vertical shaft in relation to the permanent Bench Mark on the surface.

Methods : The depth of shaft may be measured by the following methods -

(a) Direct measurement by steel band or by special shaft tape.
(b) Indirect measurement emplying a plumb wire.
(c) Measurement by modern techniques using Geodimeter, Radar apparatus, etc.

Direct Measurement by Normal Steel Band or tape : In this method, soft metal plates must be attached to the sides of the shaft at intervals equal to the length of the band or tape. The band or tape in use may by either 30m or 100m long. The longer band being preferred in deep shafts.

The metal plates must be attached to the shaft walls and this will require some preparotary work. The plates may be drilled so that small adjustable hook pegs can be inserted to suspend the tape or band at each plate. The measurement must commence at some point on the shaft wall near the top where the reduced level has already been determined. The tape is

lowered down the shaft and must be tensioned taking into consideration the weight of it. It is not attempted to make the end of the tape co-incident with the point of known level. The top end of the tape is marked on the soft metal plate with a knife-edge scratch. There will thus be a correction which may be either positive or negative which can be measured by a smaller scale.

The surveyour rides on the top of a cage, which is lowered to the lower end of the tape, where a marker plate is fixed to the shaft wall. The tension is applied and a reference mark made on the plate. The cage is then raised to the surface, the tape removed from its surface position and the cage is lowered to the reference mark in the shaft. The tape is fixed to the plate by means of an adjustable hook so that its zero coincides with the reference mark previously fixed. The cage is lowered again to the end of the tape and a similar mark is fixed on the plate attached to the shaft wall.

The process is repeated until the whole of the shaft has been measured and, whilst the tape is suspended in its final position, opposite the bottom or inset, a permanent bench mark is fixed in underground roadway by observations with a levelling instrument.

Each bay length must be corrected for temperature and for tension if it is varied from the standard tension. Care must also be taken to subtract from the total measured length an amount equal to the thickness of the handle multiplied by the number of times the tape was suspended in the shaft. This assumes that the outside of the handle forms the zero of the tape. If the zero of the tape is at a distance from the end, then an amount equal to the distance between the inner edge of the zero of the tape, multiplied by the number of suspensions must be added to the total measured length.

It is essential, that the tape is hanging free and vertical. Any deviation from the vertical may cause inaccuracies.

Good shaft signalling arrangements must be made so that clear understanding is maintained between the winder, banksman, onsetter, surveyor working on top of the cage and his assistants.

Safety harnesses must be used by the persons on the cage at all times.

The measurement of the shaft should be made at least twice and possibly in the reverse direction and the mean taken as the correct measurement, providing the required accuracy has been achieved. The degree of accuracy which may be expected is about 1 in 5,000 with careful handling of equipment.

A disadvantage of this method is that the cage must be continually raised and lowered.

Direct Measurement by Special Shaft Tape : This method involves the use of a very long steel tape, say 300 metres or, exceptionally, 1000 metres. The tape is normally wound on a large drum controlled by handles and ratchets, and positioned on the surface near to the shaft.

The tape is passed over a pulley, mounted in the headgear and lowered down the shaft

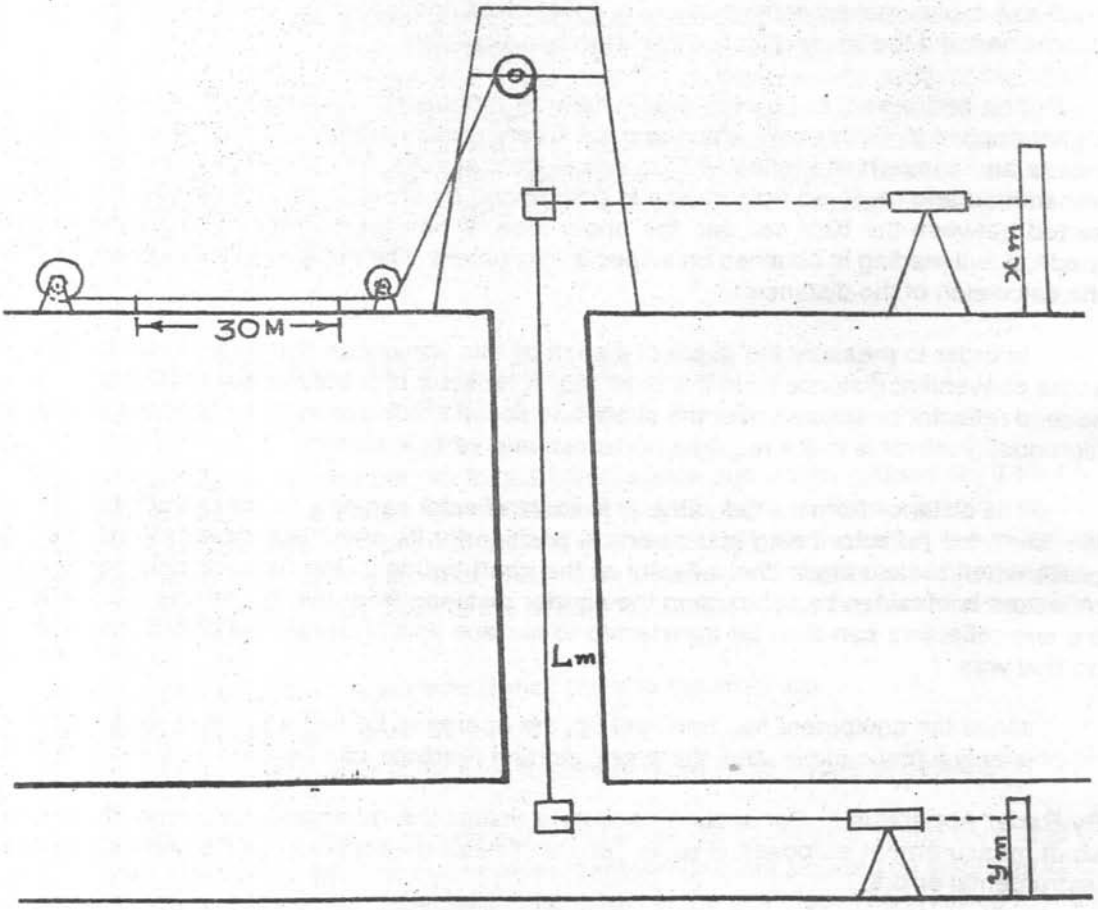
until the lower end is suspended in the sump, below the shaft bottom. Care should be taken to keep it free from obstruction during lowering down the shaft. A weight is attached to the free end of the tape, and after the tape has settled, observations are taken to it, with a level at the surface and underground. An underground bench mark is then established for permanent reference.

Correction for deviations from standard temperature and tension must be applied, and due to the length of tape employed, allowance for extension must be made for its own weight. The weight of the tape causes elongation of the tape which varies with the length of the tape.

The mean of a number of separate measurements should be taken as the accepted value after the corrections have been applied.

Indirect Measurement Employing a Plumb Wire : A steel piano wire of about 16 to 18 S. W. G of sufficient length is wound on a drum placed at about 40m from the shaft. Arrangements are made to draw the wire out horizontally and guide it round two pulley wheels on the shaft headgear so that the wire, weighted by a 2 kg plumb bob can be lowered freely to the shaft-bottom. Underneath the horizontal portion of the wire, between the drum and pulley, a base-line is established over two marking stations, exactly 30m apart. The weighted wire is now lowered to the shaft bottom and allowed to hang for 3 - 4 hours, being periodically raised and lowered to permit stretching to take place before any measurements are taken.

A mark is made on the plumb-bob and the latter raised or lowered until it coincides with the line of sight of a levelling instrument set up at a suitable point near the shaft top. When this has been done a piece of white thread is tied on the wire exactly opposite the rear mark of the base-line nearest the drum. The plumb-bob is then lowered down the shaft until the thread on the wire is exactly opposite the forward end of the base-line near the shaft. This represents the first 30m length. A second piece of thread is now tied opposite the rear end of the base. The plumb-bob is thus lowered, one base length at a time, until the mark on it is intersected by the line of sight of another level, set up in the shaft bottom. At this point a reading is taken from the level to a fixed bench mark at the foot of the shaft and the odd final length from the surface base measured with a steel tape. The total length of wire measured in this way gives the distance between the lines of sight of the levels at the surface and in the shaft bottom. Knowing the reduced levels at shaft top and floor of the shaft bottom the depth of shaft can be found out.



Shaft Measurement
(Plumb-wire over a Fixed Base)

Depth of shaft = L + Y − X metres as shown in the figure.

Errors may be made marking with the thread and in ensuring co-incidence with the lines of sight. The amount of stretch in the wire tends to vary as the length of suspended wire varies during the measuring operations. The temperature of wire in the shaft is different than the temperature on the surface and this means futher inaccuracies. The method should therefore be made several times, possibly in the reverse direction and the mean value should be taken as the correct measurement.

Measurement by Modern Linear Measuring Equipments :

By Geodimeter : The principle of Geodimerer or Geodetic Distance Meter is based on the transmission and reception of light waves.

A modulated light beam is transmitted by the instrument to a reflector, which is placed at the other extremity of the distance to be measured. This reflector transmits the light back to the Geodimeter where a phase comparison is made between the outgoing and ingoing light pulses. A measurement of the number of light pulses and fractions that occur in this distance is obtained and the linear distance can then be calculated.

The beam sent out by the Geodimeter is modulated by means of a high frequency signal applied to a Kerr cell, which acts as a very rapid electronic shutter. A photo tube receives and converts the reflected light into electric signals. The phase relation between the transmitted and received light pulses is determined by an adjustable electric delay line, inserted between the Kerr cell and the photo tube. When the correct adjustment has been made, a null reading is obtained on a special instrument. The delay line readings are used for the calculaion of the distance.

In order to measure the depth of a shaft by this apparatus, the Geodimeter is set up at some convenient distance from the shaft top. A reflector is placed at the shaft bottom and a second reflector positioned over the shaft. The second reflector must be oriented so that the high quality mirror is in the required horizontal and vertical planes.

The distance from the Geodimeter to each reflector can now be measured, the mirror of the shaft top reflector being in the vertical position for its own distance and tilted at 45 degrees when measuring to the reflector at the shaft bottom. The distance between the two reflectors is obtained by subtracting the shorter distance from the longer one. The levels of the two reflectors can then be transferred to surface and underground bench marks in the normal way.

Once the equipment has been set up, the operating times for a single shaft measurement is only a few minutes and, therefore, several readings can be taken in a short period.

By Radar Apparatus : For securing accurate results the radar apparatus may be used for shaft measurement purposes. It gives results of extreme accuracy and eliminate human or instrumental errors.

Radar beams underground are usefully applied on straight and level roadways. The science of radar can very well be adopted to measure the depth of shaft which is of necessity straight and vertical and where radar beams can be projected directly from the surface to the bottom of the shaft.

A radar projector apparatus (short beam type) would be fixed over the shaft mouth (the top cage being elevated into the headgear to give more room) and the beams directed vertically down the shaft. At the bottom of the shaft, near the centre and level with the shaft inset floor, a "target" is fixed consisting of a flat disc 0.6m to 0.9m in diameter and 2.5cm thick, of laminated tinselled paper, (Tinsel presents good radar beam refraction). The radar beams projected down the shaft to the target are refracted back to the apparatus, amplified and recorded on a radar screen which will be calibrated to read in terms of distance in metres.

In order to obtain accurate readings, radar refractions due to the steel in the shaft, such

as rope, guides, joists and cage, etc, are eliminated by the recording of the beams refracted before placing the target at the shaft floor. The recorder will then be set to "zero". When this is done, the projector will again be operated and the beams refracted now will be those due to the refractions set up by the tinselled target alone, and will record the distance between the centre of the projector to the centre of the target at the floor level of shaft bottom. If the centre of the projector be made level with the level of the shaft mouth the accurate depth of the shaft will be recorded. Where the instrument or apparatus is carefully "tuned" accuracies up to the standard of one mincron error will be attained. This result and standard of accuracy can not be obtained by any other known method. Other techniques could be applied to ascertain shaft depths, such as photometric cell and the supersonic technique, but the accuracies attained do not compare with the radar technique.

LEVELLING TO ESTABLISH UNDERGROUND BENCH MARK

Mines Regulations require that all levels taken above and below ground shall be referred to a plan in relation to a Bench Mark established on the surface. Sometimes it may be required to establish the level of a point at bottom of a newly sunk vertical shaft in relation to the permanent Bench Mark on the surface. Such work requires maintenance of high degree of accuracy. To establish a Bench Mark at the bottom of a shaft the survey involved consists of three parts :

(a) Flying levelling from a surface Bench Mark to the shaft top.

(b) Measuring the depth of shaft.

(c) Flying levelling from shaft bottom to underground Bench Mark at desired point.

SURFACE LEVELLING : This is carried out using a dumpy Level or a Tilting level and a tested staff, starting at the nearest available permanent Bench Mark situated on stable ground and ending at another bench mark specially established at the shaft top. The levelling instrument should be checked for collimation error before commencing the levelling and should be placed in the middle of the back and fore sights and levelled perfectly. The staff should be provided with spirit bubble or a plumb bob for checking its verticality during observations. Sights longer than 50 metres should be avoided to ensure good optical definition of the staff. All levelling operations should be carried out in the close circuit to check the error and if the closing error is more than the desired value the levelling operation should be repeated.

MEASURING DEPTH OF SHAFT : The depth of shaft may be measured by the folowing methods -

(a) By using a special long steel tape and reading direct at top and bottom of shaft simultaneously with necessary correction for stretch due to temperature and weight.

(b) By using an Electronic Distance Measuring device, the most accurate method

using a **Geodimeter** (Geoditic distance meter) which measures the time taken for a beam of light to travel between two fixed points.

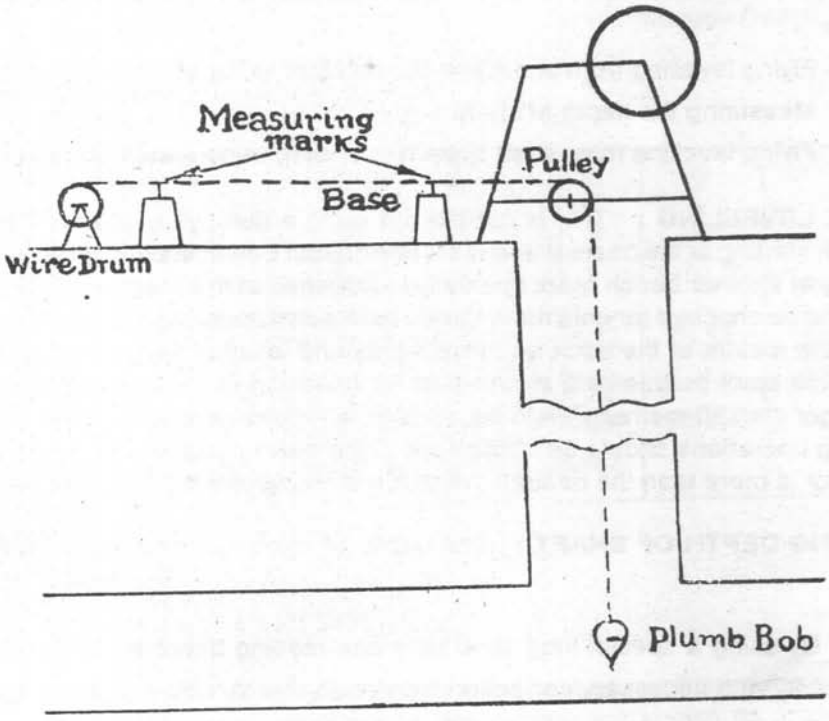
(c) By lowering a weighted wire and measuring its lengths.

The simplest method is by direct measurement with a long steel band to cover the depth of shaft, but this is not always available, therefore, in absence of an E. D. M. device, shaft measurement should be carried out **by means of a plumb wire over a fixed base.**

Equipment Required :

1. Steel piano wire 16-18 S.W.G. of sufficient length to cover the depth of shaft and the surface base, etc.
2. One plumb bob weighing 14-16 kg made of lead, and one 2 kg weight.
3. Two levels with staves.
4. Wire drum with operating handle.
5. Pulleys for suspending plumb wire and for guiding the wire over fixed base
6. Two marking tables.
7. Steel tape.
8. Thread, note book, pencils, etc.

Procedure : The steel wire is wound on a small drum, fitted with handles for rotation and with a pawl and ratchet for better control, sited on the surface at about 40m from the shaft. A pulley



Determining Depth of Shaft

is fixed over the shaft at the same level as the drum, so that the wire can pass horizontally from the drum to the pulley over two marking stations, exactly 30m apart, serving as the *Base Line*. Two pulleys are sometimes used, one being at the shaft top to pass the wire from the drum horizontally over the marking stations and the other pulley at some convenient upper level of headgear so that the wire can be lowered freely at the shaft bottom. The distance between the marking stations is carefully measured by steel tape after due consideration to standard length of the tape and temperature.

After passing over the pulley the wire is lowered down the shaft with a small weight (2 kg) attached to it which is replaced by a plumb bob weighing 14 - 16 kg when the wire is below the shaft bottom. The wire is allowed to hang freely for several hours being periodically raised and lowered again to permit stretching to take place before measurements.

The wire is wound up to the shaft top and a mark is fixed on the plumb bob and it raised or lowered until it coincided with the line of sight of the level set up near the shaft top. A piece of thread is tied on wire exactly opposite the rear mark (near the wire drum) of the base line. The plumb bob is lowered down till the thread on wire is exactly opposite the marking table near the headgear to measure 30m of depth. A second piece of thread is tied opposite the rear mark and the blumb bob is lowered again till it coincided with the other mark thus measuring 2 x 30 or 60 metres of depth.

In this manner, the plumb bob is lowered one base length (30m) at a time until the mark on it is intersected by the line of sight of another level set up in appropriate inset in the shaft bottom. Readings are taken from this level to a bench mark in the inset. The odd length of the wire at surface base, from the last thread on the wire to the rear end of the base iine, would be measured by a steel tape.

The total length of the wire measured off in this way is the distance between lines of the sight of the levels at the surface and in the inset.

Distance between Bench marks = (N x 30) + (Odd length) + (Underground staff reading) – (Surface staff reading)

Where N = Number of full 30m lengths counted.

Knowing the reduced levels at shaft collar and floor of shaft bottom the depth of shaft can be found out.

The method will give quite good results provided care is exercised, particularly at the marker stations. The amount of stretch in the wire tends to vary as the length of suspended wire varies during the measuring operations. The measurement of shaft should, therefore, be made at least twice and possibly in the reverse directions and the mean should be taken as the correct measurement.

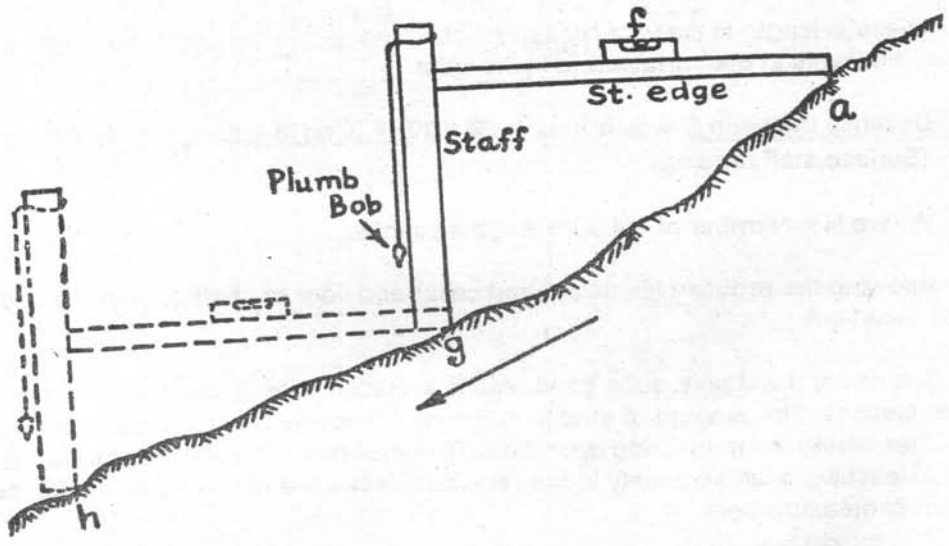
UNDERGROUND LEVELLING : A flying level is carried out from the temporary bench mark at the shaft bottom to the permanent bench mark previously established.

ACCURACY : The survey work will be considered satisfactory if the limits of error are within the following ranges -

Surface Levelling	:	Not to exceed 2cm per km.
Shaft Measuring	:	Not to exceed 1 in 5000 of depth measured.
Underground Levelling	:	Not to exceed 4cm per km.

LEVELLING WITH STRAIGHT-EDGE

The usual forms of level and staff may be used belowground in precisely the same way as they are used on the surface, except that the sight will generally be much shorter. When the gradient exceeds 1 in 5 it becomes exceedingly difficult to do good work on account of the short shots obtained at each setting of the instrument and the difficulty of focussing the telescope on the staff at short range. When the gradient is moderate, say 1 in 10, it is hardly possible to get sights more than about 12 metres long in roads 1.8 metre high. The levelling for steep inclined roadways cannot, therefore, be done satisfactorily with the dumpy level, and recourse must be had either to trigonometrical levelling or straight-edge levelling. The trigonometrical levelling necessitates the use of an expensive instrument and a knowledge of trigonometry, whreas the straight-edge levelling only requires instrument that can be readily obtained and calculations of simplest kinds are involved. The straight edge levelling gives only approximate results.



Levelling with straight-edge

The straight-edge or stave should be well-seasoned timber about 2.5cms wide and 5 to

8cms deep. Its length depends upon the gradient and height of the roadway. The steeper the roadway and the less the available height, the shorter must be the straight-edge in the length. Thus if the roadway dips 1 in 2 and its height is 2 metres, the maximum length of straight-edge would be 4 metres. But this would be inconvenient in length. So the useful length of the straight edge should be 2 metres. Other accessories and equipments required are spirit-level, plumb-bob, levelling staff, steel tape, etc.

As the accuracy of the levelling with a straight-edge depends on the stave, it must be checked for length, straightness and parallelism of the edges. The accuracy of the spirit-level must also be tested. **The length** of the straiget-edge must be compared with a standard steel rule or steel tape in good condition. **To test for straightness,** lay the stave flat on a plane surface and draw a fine line close to one edge. Turn the stave over with the other flat side uppermost (not end for end) and adjust the ends of the same edge carefully to the end of the line. Draw another line along this same edge. If the stave is truely straight, the two lines will coincide, But if it is curved or crooked, there will be two lines instead of one and the extent of inaccuracy will be shown doubled. The other end should then be tested in a similar manner. **To test for parallelism,** lay the straight-edge or stave first flat on a smooth table and draw the out line of it with an H. B. pencil. Now turn the stave end for end and apply the upper edge to the upper line as at first. Draw a second line along the lower edge. If two edges are parallel, the two lower lines will coincide. If not, these will intersect. A callipers or any suitable gauge may also be applied to take the width of the stave at each end. If these are alike the edges are parallel. **To check for accuracy of the spirit level** bring bubble to the centre of its run, turn the level end for end and observe if the bubble returns to the central position.

Before commencing the levelling the length and depth of the straight-edge should be carefully measured. The plumb-bob should be attached to the levelling staff. One end of the straight-edge is placed at the starting point 'a' and the other end of the straight-edge is elevated or raised until the bubble in the spirit level 'f' placed on the stave is in the middle of its run. The surveyor's assistant next brings the staff in position until it stands vertical with the graduated face just touching the end of the straight-edge. When the staff is correctly set the plumb-bob should hang parallel with the edge of the staff. The reading of the upper edge of the straight-edge on the staff is now booked as a fore sight and the depth of the stave as back-sight. The point 'g' is marked on the ground and the straight-edge and the staff are moved to the new positions 'g' and 'h' respectively. In its new position, one end of the straight-edge is placed at 'g' and other end is raised or lowered until it is horizontal. The staff is agian held vertically opposite the face with hanging edge of the stave, the reading taken and the point 'h' is marked on the ground. The reading is booked as the foresight, the length of the stave as the horizontal distance and the depth as the back sight.

It will be noted that the back sight is constant as the depth of the straight-edge. Had the travelling being done uphill, i.e. upward direction, then the for sight would have been constant.

Booking : Length of straight-edge = 2 metres. Depth of straight-edge = 8cms.

No. of Setting	B. S.	F. S.	Rise	Fall	R. L.	Horizontal Distance	Remarks
	0.08				150.00	0	R.L of Starting point - 150.00 metres.
1	0.08	2.08	-	2.00	148.00	2	
2	0.08	1.96	-	1.88	146.12	4	
3	0.08	2.12	-	2.04	144.08	6	
4		1.76	-	1.68	142.40	8	
	0.32	7.92	0.00	7.60	150.00		
		0.32		0.00	142.40		
Check		7.60		7.60	7.60		

TRIGONOMETRICAL LEVELLING

When the inclined distance between two points is known the relative altitudes and the horizontal displacement of the points may be determined by reading with the vertical circle of the theodolite the angle of elevation or depression of the line joining the points. This method of finding altitudes is known as Trigonometrical Levelling. This method is suitable in steep gradients where the dumpy level is inconvenient to set and where extreme accuracy is not required, only an approximate difference in level between two points is urgently required.

Equipment required : A theodolite or a modern mining dial, levelling staff, steel band or tape, plumb-bob, field book, pencil, etc.

Procedure : Let the vertical displacement of two points P and Q is required. The instrument is set up carefully over station P and leveiled. The height of the axis of the telescope above the floor is measured. The staff is held by an assistant on station Q and the telescope is directed to a mark on the staff at the same height as the instrument at P. The vertical angle is noted. The measurement of the angle is repeated on the other face of the instrument. The mean of the readings is taken as correct. The inclined distance between P and Q is also measured for long sights the inclined distance may be measured by means of stadia readings.

Now if the distance along the slope = PQ

the vertical angle = θ

Then the vertical displacement = PQ x Sinθ

the horizontal equivalent of PQ = PQ x Cosθ

In this method the instrument is set up at alternate station for subsequent readings. To ensure maximum accuracy, the vertical angles must be measured at least twice on each face, the altitude bubble being carefully centered at each pointing. A sliding target should be fitted to each staff so that the correct point can be easily distinguished.

Advantages : The trigonometrical levelling has the following advantagtes over the ordinary levelling with a dumpy level -

(1) It is improtant in the underground working of seams inclined at angles of 10 to 12 degrees or upwards, where the use of ordinary level would be inconvenient by reason of the shortness of sights for restricted heights.

(2) This method of levelling is suitable for levelling hilly and mountaneous regions where ordinary levelling with a dumpy level is difficult and time taking.

(3) It is preferred for levelling steep gradients and when distances involved are large.

(4) In conjunction with stadia measurements this method is used for contouring in hilly areas.

(5) As the inclined length is measured along the floor any error in measurement due to tape sagging can be avoided.

Disadvantages : (1) The degree of accuracy attainable by this method is low in comparison to that attainable by levelling with a dumpy level. The *Permissible error for trigonometrical levelling is 1.5cm per 1 km of run and for ordinary levelling with a dumpy level it is 0.15cm per km of run.*

(2) Profile of the ground may not be parallel with the inclined line of sight. The linear measurements along the ground in such cases can never be equal to the inclined distances.

(3) The accuracy is entirely dependent upon the linear measurements, vertical angles and the ground profile.

(4) The method is preferred only when extreme accuracy is not required.

Example 35. An instrument was set up at a distance of 180m from a church tower and the angle of elevation to its top was 2°42'. The staff reading on a Bench Mark of R. L. 54.225m with telescope horizontal was 0.880m. Find the reduced level of the top of church tower.
(Survey Comp. 1963).

Solution : Reduced level of the horizontal line of sight = 54.225 + 0.880 = 55.105m.
Elevation of tower of the church above the line of sight = 180 x tan 2°42 = 8.490m.
Hence, R. L. of the top of church tower = 55.105 + 8.490 = **63.595m.**

Example 36. An instrument was set up at P and the angle of elevation to a vane 4m above the foot of the staff held at Q was 8°30'. The horizontal distance between P and Q was known to be 2140 metres. Determine the R. L. of the staff station Q, given that the R. L. of the instrument axis was 2450.45.

Solution : Height of the vane above the instrument axis = D tanθ = 2140 x tan 8°30' = 319.825 metres.

Correction for curvaure and refraction	=	0.0673 D^2 metres, when D is in km.
Hence, correction = 0.0673 $(^{2140}/_{1000})^2$	=	0.308m (+ve).
Height of vane above instrument axis	=	319.825 + 0.308 = 320.133m.
R. L. of vane	=	320.133 + 2450.450 = 2770.583m.
R. L of the staff station Q	=	2770.583 – 4.000 = **2766.583m.**

Example 37. The following notes were obtained from an underground trigonometrical levelling. The level of point A is 41.50 metres above datum. Calculate the level of point E. Values are in metres. .

1st Stn	*Height of Instrument*	*Staff Stn.*	*Staff Reading*	*Vertical Angle*	*Measured. Slope Dist.*
B	1.35	A	1.35	–18°16'	114.00
-	-	C	1.35	+10°15'	92.20
D	1.57	C	1.57	+15°30'	109.60
		E	1.57	+17°48'	96.40

(Survey Comp. 1971)

Solution : The calculations are tabulated as follows -

At Stn.	Ht. of Inst.	Staff Stn.	Staff Reading (M)	Vert Angle (θ)	Slope Distance (L)	Hor. Dist L x Cosθ (D)	Vert. Comp L x Sinθ (V)	Ht. of coll RI. V + M (H. + C.)	R. L. HC + V M
B	1.35	A	1.35	– 18°16'	114.00	108.26	– 36.73	78.58	41.50
B	1.35	C	1.35	+ 10°15'	92.20	90.73	+ 16.41		93.64
D	1.57	C	1.57	+ 15°30'	109.60	106.61	+29.29	65.92	93.82
D	1.57	E	1.57	+ 17°48'	96.40	91.79	+29.47		

CONTOURING

The value of a plan or map is highly enhanced if the relative position of the points is represented both horizontally as well as vertically. Such maps are known as *topographic maps.* Topography may be represented on a map by hachures or hill shading by forming lines,

by tinting, or by contour lines. *Hachures* are a series of short lines drawn in the direction of slope. For a steep slope the lines are heavy and closely spaced. For a gentle slope they are fine and widely spaced. Hachures are used to give a general impression of the configuration of the ground, but they do not give the actual elevations of the ground.

Contour Line : A contour line is the intersection of a level surface with surface of the ground. It is an imaginary line connecting the points of equal elevation above or below a given datum. A contour is represented in nature by the shore line of a body of still water. Since the contour lines on a plan or map are drawn in their true horizontal positions with respect to the ground surface, a plan containing contour lines shows not only the elevations of points on the ground, but also the shapes of the various topographic features, such as hills, valleys, ridges, etc. Contours give a maximum amount of information without obscuring otner detail portrayed on the map or plan.

Contour Interval : The vertical distance between any two consecutive contours is called the contour interval. The contour interval is kept constant for a contour plan, otherwise the general appearance of the plan will be misleading. The horizontal distance between two consecutive contours is known as the *horizontal equivalent*. The horizontal equivalent depends upon the steepness of the ground. The choice of proper contour interval depends upon the following considerations -

(a) *The nature of the ground :* For a flat ground a small interval is necessary. If the ground is steep or broken, a greater contour interval should be adopted, otherwise the contours will come to close to each other.

(b) *The scale of the plan :* The contour interval should be inversely proportional to the scale. If the scale is small, the contour interval should be large. If the scale is large, the contour interval should be small.

(c) *The purpose and extent of the survey :* The contour interval largely depends upon the purpose and extent of the survey. For accurate earth-work calculations, small contour intervals will be helpful. Where the extent of survey is large, the contour intervals should also be large.

(d) *Time and expenses of field and office work :* If the time available is less, greater contour interval should be used. If the contonr interval is small, greater time will be taken in the field work and in plotting the same.

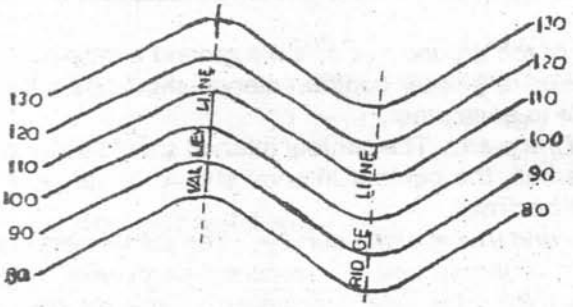
Stratum Contours : These are contour lines drawn on the surface of a bed or coal seam and are the same as the strike lines. In case of uniformly sloping bed or seam these lines are equally spaced and may form straight lines if the direction of dip is constant in the area.

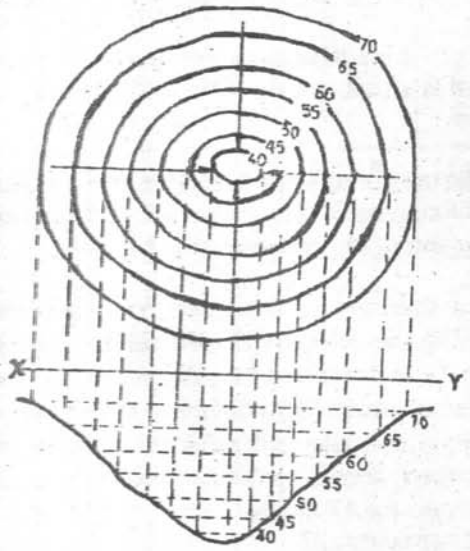
Isopachytes : These are lines showing equal vertical thickness of strata, say, between the floor of two seams. Such lines are obtained by showing the contours of two seams on the same plan in correct relationship to each other. Where these contours of the seams cross one another, the thickness of the intervening strata is obtained simply by deducting the value of the contour of the lewer seam from that of the upper seam. If one joins the points where these differences give the same value, a curved line is produced, known as *Isopachyte Line*.

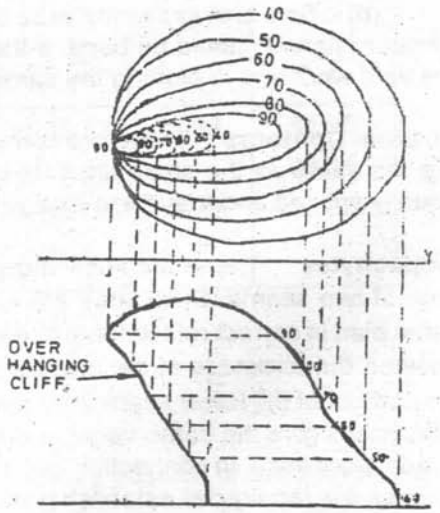
Grade Contour : In connection with the preliminary survey for a road in a hilly or mountanous country, it is required to establish points along the given gradient. The line joining such points is called a *grade contour* or a *contour gradient*.

CHARACTERISTIC OF CONTOUR LINES : The following are the characteristics of the contour lines -

1. All points in one contour line have the same reduced level.
2. Every contour line closes on itself, either within or beyond the limit of the map.
3. Contour lines are equally spaced when the ground is uniformly sloping, and where the ground is plane they are straight and parallel.
4. Contour lines run close together near the top of the hill, representing very steep ground, and wide apart at the flat ground.
5. Contours never split, nor do two contour run into one, nor cross each other, except in the rare instance of an overhanging cliff.
6. Contour lines cross ridge lines or valley lines at right angles. A ridge line is showin when the higher values are inside the loop or bend in the contour, while in the case of a valley line, the lower values are inside the loop. The same contour appears on either side of a ridge or valley.
7. A series of closed contours on the map indicates a depression or a summit, according as the lower or higher values are inside them. Depressions between summits are called saddles.
8. The direction of the steepest slope at a point on contour is at right angles to the contour.







USES OF CONTOUR LINES ON MINE PLANS : The following are the some of the important uses on mine plans -

1. Contour lines convey when delineated upon a plan, an approximate representation of configuration of the surface of the ground, the degree of approximation depending on the proximity of contours. Any change in level, flat or inclined seams, anticlinal or synclinal structures, domes or basins, etc. can be easily ascertained from a contour map.
2. A contour line is a line of strike and the direction and amount of dip can be calculated.
3. Work planning scheme can be drawn up with due regard to the dip of the seam.
4. Proper position of shafts, inclines, etc. can be located so as not to be affected by highest flood level.
5. Gradient of proposed road or railway lines at surface or haulage road below ground can be determined.
6. Static pumping head can be determined at any point.
7. Layout of stowing pipe range can be planned.
8. Area liable to flooding can be seen at a glance, and proper drainage scheme can be drawn up.
9. Intervisibility of two stations can be ascertained from a contour map.
10. The thickness of cover over a coal seam can be determined from stratum contour or underground levels and surface contour lines. Isopachytes can be found out for use in quary works.
11. Quantity of earthwork can be computed.
12. Capacity of a reservoir can be determined.

METHODS OF CONTOURING

In general, there are mainly two field methods of determining contours -

(a) Direct Method.

(b) Indirect Method.

In each method, the location of a point involves both horizontal as well as vertical angle.

Direct Method : The method in which contouring of the points of required elevation are directly located on the ground with the help of levelling instrument is called Direct Method of contouring. The position of these points are surveyed by chain and offset method or by a plane table. The contours of required elevations are drawn by joining the respective point. The method is accurate, but it is slow and tedious as lot of time is consumed in tracing the points of the same elevations on the ground.

Contouring by direct method is done by any of the following mehods :

1. By selecting a long main line and then taking cross-sections at suitable intervals.
2. By radial line method.
3. By use of plane table in conjunction with levelling operation.

Indirect Method : The method in which spot levels taken on already fixed points over the entire area, their respective R.Ls written against each point on the plan drawn to scale and contour lines are drawn by interpolation is called *Indirect Mehod of Contouring.*

In this method the spot levels are taken on points fixed along series of lines laid out over the area to be contoured. The spot levels so taken are not necessarily on the contour lines. Spot levels at several representative points, representing ridge and valley lines, hills and depressions, and important changes in the slope scattered all over the area are observed. The position of points are then plotted to scale on the sheet where the elevations written against each point. The contours of required elevation are then drawn by interpolation. The method is used in all kind of surveys being cheaper, quick and less tedious compared to direct method of contouring.

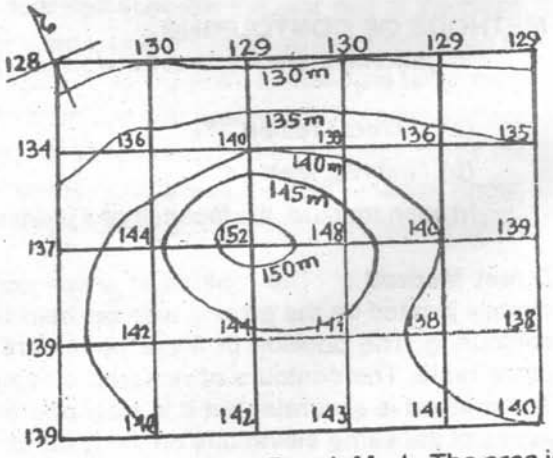
Contouring by indirect method is done by any of the following methods :

1. By square method.
2. By cross-section method.
3. By tacheometric mehod.

CONTOURING ON FLAT GROUND

In this method a series of levels is obtained irrespective of lthe contour height required. These are usually taken at a predetermined horizontal inrervals along lines of known direction. These intervals will depend upon the extent of the area, nature of the ground and the closeness of the vertical interval of the contour.

Instruments Required : Dumpy level, levelling staff, wooden pegs, chain, steel tape, arrows, ranging rods, theodolite or dial.



Procedure : The simplest method in this case, is the method of unit squares - an *indirect method.* The whole area is first marked out in squares of convenient size, the side of which may vary from 3m to 30m depending upon the nature of the ground, the contour interval and accuracy of the work required. The corners of the squares are marked with pegs with the help of a dial or theodolite and steel tape. Each peg is numbered so as to facilitate plottiog their positions afterwards.

The levels of all the pegs, termed spot levels, are ascertained by levelling with a dumpy level and a staff. and are correlated to any nearby permanent Bench Mark. The area is

then surveyed with the theodolite or dial so that the positions of the pegs may be plotted to scale on a plan. Reduced levels of all the pegs are calculated and marked on the plan.

The contours may now be drawn by inspection as smooth curves through all points of equal level. Sharp angles should be carefully avoided. The contour line may be found by a process of interpolation involving proportional judgement by eye or arithmetical calculation.

CONTOURING ON STEEP GROUND

In this case the foregoing method is not suitable because of the difficulty of squaring out the ground, and it is more usual to adopt a method of *direct location of contours*, especially where the area is small and considerable precision is required.

The method involves -

(a) Locaing a number of points on each contour and marking their positions by pegs.

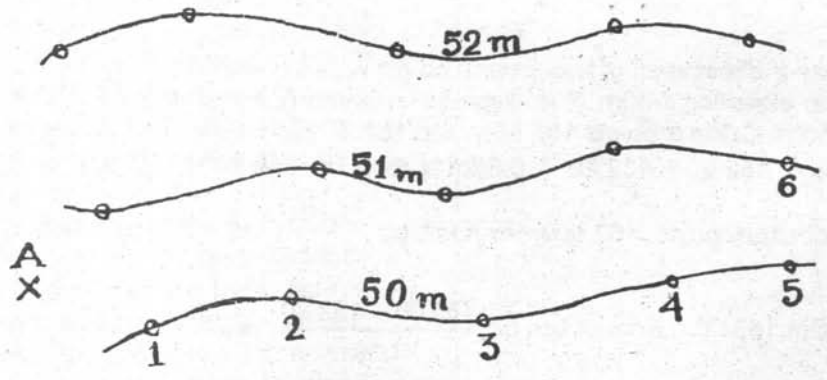
(b) Making a dial traverse along each line of pegs to enable the contours to be plotted.

Assuming that it is required to peg out the 50m contour, the first step is to level from the nearest Bench Mark to some point A, where the instrument is placed to command as much ground as possible. It is usual to book by the height of collimation method and the instrument height being known (Let 51.500m). The staffman would be instructed to move up or downhill until the reading on the staff was 1.500m. so fixing peg 1 on the 50m contour. In a similar way, the staffman would be directed to pegs 2, 3, 4 and 5 at which the staff readings for being these 1.500m.

Thereafter a number of pegs on the 51m contour may be fixed, the staff readings for these being 0.500m.

When as much ground as possible had been covered from A, the instrument would be moved to another point and a new "collimation height" established from a backsight on to a peg of known reduced level. This would enable more points on the 50m and 51m contours to be located, after which 52m contours may be tackled, and so on.

When all the required pegs have been inserted a dial and chain traverse would be carried out along each line of pegs. The points would then be plotted to scale on a plan and smooth cuves drawn through them to represent the contours.



CONTOURING BY TACHEOMETRIC METHOD

In this method the instrument known as *tacheometer* is used. A tacheometer is a the-

odolite fitted with a diaphragm having two stadia wires, one above and other below the central wire. The staff intercept is then calculated by taking the difference between the readings of the upper and lower stadia wires. Horizontal distance is determined by multiplying this intercept with the stadia constant of the instrument which is usually 100. The horizontal distance thus need not be measured, since the tacheometer provides both horizontal and vertical control.

This mothod is most suited when a contour map of a hill is required. A tacheometer can command more area and thus it is cheap and economical comparatively. From each of the traverse station a number of radial lines are set out at a known angular interval and pegs are driven on these radial lines as per requirement.

The elevation and the horizontal distance of each point is determined and recorded. The survey is plotted on the plan and the contour lines are then interpolated as usual.

METHODS OF INTERPOLATING CONTOURS

Interpolating of contours is the process of spacing the contours proportionally between the plotted ground points which have been established by indirect methods of contouring. For interpolating a contour it is assumed that the ground between any two points is uniformly sloping. There are three methods of interpolating contours :

1. By Estimation Method.
2. By Arithmetical calculation Method.
3. By Graphical Method.

1. By Estimation Method : The method in which the positions of contour points between ground points are estimated roughly and the contour lines are drawn through these points is called estimation method.

If the elevations of two points A and B are 152.20 and 153.50, thus having a difference in elevation 153.50 – 152.20 = 1.30m, the postiion of contour points 152.50m and 153.00m between A and B can be marked roughly by estimation considering the difference of elevations as a horizontal distance between the points dividing the distance between A and B proportionally. This method is rough and it gives for small scale work only.

2. By Arithmetical Calculation Method : In this method the interpolation of contours is done by mathematical calculations. The method is time consuming but it gives accurate results.

Suppose the elevation of two ground points A and B are 152.20m and 153.50m having a difference in elevation 1.30m. The distance between A and B is say 25m and the contour interval is 0.50m. Contour points 152.50m and 153.00 can be located between A and B. Now the difference of 152.50 – 152.20 = 0.30m is for a contour point 152.50 from A. Hence the distance of contour point 152.50 from A will be $\frac{0.30}{1.30} \times 25 = 5.77$m. Similarly the distance of contour point 153.00m from A will be $\frac{153.00 - 152.20}{1.30} \times 25 = 15.38$m. These distances are plotted to scale on the map, the scale being kept the same as that of the map already drawn.

3. By Graphical Method : The method in which interpolation of contour is done by use of graph paper is called Graphical Method. There are several graphical methods for interpolating contours from spot levels. The methods are simple and accurate.

TRACING OF OUTCROP ON CONTOUR PLAN

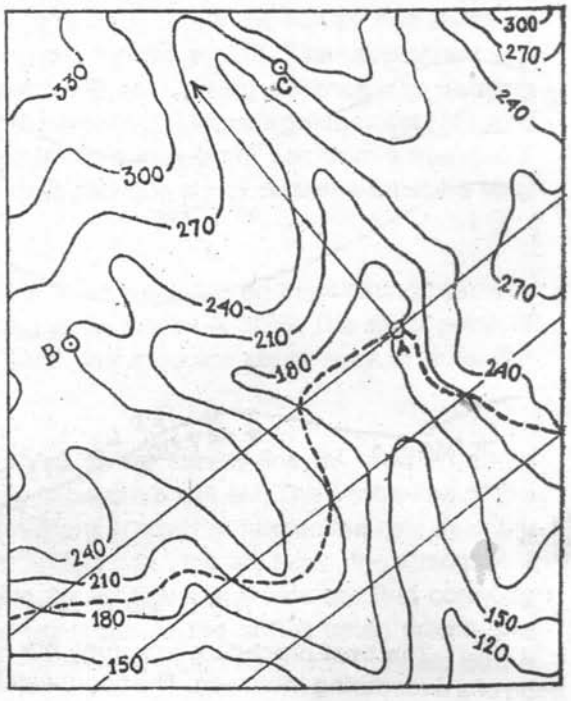
Outcrop of a coal seam may be traced on a contour plan when any seam outcrops at a point and the depths of the same seam from the surface at different points are known.

Example 38. Attached is a contour plan of an area wherein a coal seam outcrops at A. Two broeholes drilled at B and C encounters the same seam at the depth of 120m and 180m respectively. Trace the outcrop of the coal seam in the plan enclosed. Explain the manner in which the outcrop is traced. **(Survey Comp. 1981)**

Solution : Datum level of seam at B = 240 – 120 = 120m
Datum level of seam at C = 270 – 180 = 90m
Horizontal distance AB measured = 408m
Horizontal distance AC measured = 372m

Fall from A to B along coal seam = 180 – 120 = 60m
Fall from A to C along coal seam = 180 – 90 = 90m
Gradient of coal seam along AB = 80 in 408 = 1 in 6.8
Gradient of coal seam along AC = 90 in 372 = 1 in 4.1

AB and AC are joined, 6.8 units along AB and 4.1 units along AC from A are measured. The line joining these two points is the strike line of the coal seam. Direction of full dip of the coal seam, at right angle to the strike line, is shown by the line with arrow head and the amount of dip appears to be about 1 in 4.



Tracing of Outcrop

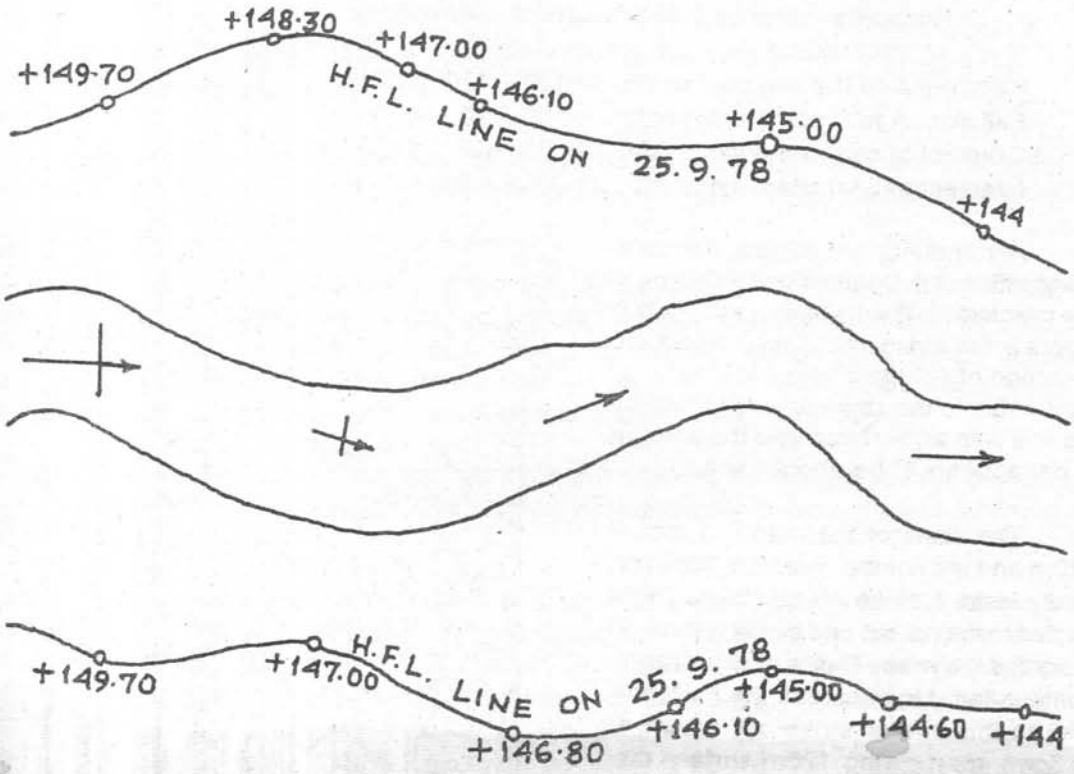
The scale of the map is 1 inch = 120m and the contour lines are 30m vertically apart. A strike line is drawn at right angles to the full dip and passing through A across the map. This is 180m stratum contour line. Line accross the map parallel to this line of strike are drawn 1" (2.5cm) apart giving 120m surface distance on the map and stratum contours 30m apart vertically as the seam dips at 1 in 4. To the rise of the seam the next line from the 180m line is the 210m stratum line because it represents the plane of the seam at 30m higher level than the

point of outcrop A. On the dip side the stratum contours are 150m. 120m, etc. but their surface contours are on the dip side and, therefore, can be no outcrop. Where a stratum contour intersects a corresponding surface contour line, the point is marked. The seam outcrops there. These points are joined by a scattered line which is the outcrop of the seam.

HIGHEST FLOOD LEVEL LINE

The perimeter of river water during maximum flood is called the *Highest Flood Level line* or simply H. F. L. line. The perimeter of water in a lake or pond is a contour line when it is at rest. But the water of river is not at rest, it is flowing under current therefore the H. F. L. line is not a contour line. It must be shown on the surface plan of the mine to avoid inundation.

To ascertain the H. F. L. line of a particular river or jore the following procedures are adopted :



H. F. L. line of River Nunia near S. J. Colliery

(a) The best procedure to survey the H. F. L. line is to attend and witness a highest flood of a river during monsoon. The flood water carries grasses, weeds, etc. and deposits the same at curved edges of its two banks. Erosion may also be found and layers of river clay or

silt are deposited showing the maximum rise of water of flood. The impressions kept by the flood water on the ground can easily be recognised which indicates H. F. L. line.

Pegs are driven at suitable positions on the impression line giving maximum rise of flood water and a survey is made from at least two permanent survey stations in the area. A theodolite traverse should be carried out following the course of H. F. L. line taking position of pegs by offsets. Reduced levels of all the pegs in relaion to line with R.Ls of lthe pegs should be plotted on the surface plan of the mine with date on which the flood level reached the line. All the peg marks should be made permanent which will help to indicate any further rise of water in subsequent flood during monsoon afterwards.

(b) When the establishment of H. F. L. line is to be made long after the flood, local information may be collected for the same. The most reliable persons who might have witnessed the flood or might be knowing the situation may be interviewed and from their informations the H. F. L. line can be ascetairned after usual survey in the area.

MAINTENANCE OF DIRECTION AND GRADIENT OF A DRIFT

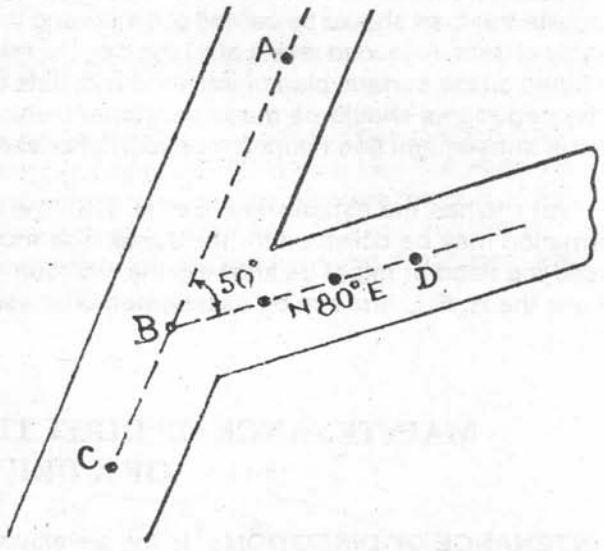
MAINTENANCE OF DIRECTION : In the development of a mine it is often necessary to set out long important roads from the ends to meet in line and, if they are cross-meaure drifts, at predetermined gradients. Such a project requires a careful theodolite survey and levelling along existing roads between the ends of the proposed road or drift. The rectangular co-ordinates of the points in the survey are calculated and the closing distance and bearing computed as being those of the projected drivage. The gradient is found from the data obtianed by the levelling over the same course. The proposed road may be driven from a theodotite station or even between such points, and in this case the computed distance must be very carefully measrued out in the drivage.

Suppose a drift is to driven ia direction N 80°E from a point B on the existing roadway ABC in a direction S30°W, in which case the bearing of BA will be N 30°E. The exact point 'B' from which the drift is to be driven having been fixed, the men are set to work to drive 3-4 metres in the probable direction.

A dial or theodolite is set up at B, in alignment of the survey line BA, and an angle 50°00'00", the proposed angle at which the drift has to be driven, is set. The centre line of the proposed drift along the line of the sight of the instrument is now marked on the roof by chalk and cotton string. Roof plugs are put in from which weighted string are hung, the attachment to the plugs being through metal pins knocked into them. The metal pins are first correctly lined by the instrument and knocked in and again adjusted. As the drift is being driven, the centre line is extended in day to day operations by chalked cotton string with the help of the metal pins.

In order to achieve high degree of accuracy the centre line is checked with a theodolite at an interval of progress of about 15 metres.

In addition to the above method a direction light indicator is also suitable for the purpose. It is an instrument designed to set the direction of mine workings. The light spot mark of the indicator is made with the aid of an optical unit, which comprises a source of light (electric bulb) capacitor, diaphragm and objective. The light spot is projected into the face in the form of two intersecting light rays. The instrument is threaded into the roof bar 10-12m off the face. It is set in the requisite direction against the plumb lines from the three metal pins previously established with the theodolite. The instrument is centered under one of these points and directed over two other points by means of light ray.



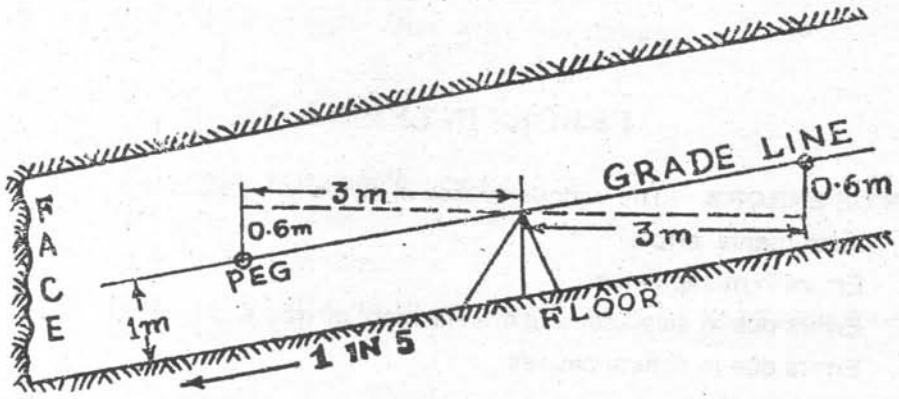
Setting out Direction

Laser (an acronym for Light Amplification by Stimulated Emission of Radiation) has proved to be a versatile working tool in mine surveying. *Laser alignment device* is now being used in India for various purposes. A low-powered laser, having been flame-proofed for attachment to adjustable brackets in fast advancing headings serves for alignment drivages. The obvious advantage being that a heading team need not stop work for placing the centre line marks, spot of red light being constantly present in the middle of the face.

MAINTENANCE OF GRADIENT : The gradient of a drift is maintained using a Dumpy level or a theodolite. In drift drivage at a fixed gradient, a grade line is established on the sides of the drift in white wash at a fixed distance usually 1 metre above the floor. Two or three pegs are inserted on the side wall at suitable intervals along the grade line.

The theodolite is set up into the drivage on firm ground and the temporary adjustments carried out. The telescope with its line of collimation horizontal is sighted back near the last peg of the grade line. A plumb string is suspended from this peg and the point out by the line of sight is established on the side wall. The distance from the point to the grade line is measured and this gives the relationship of the line of collimation of the instrument to the grade line. If the gradient of the drift were 1 in 5 dipping a distance of 3 metres would be measured as level as possible along the side towards the face. A mark would then be sighted at this distance using the theodolite. In 3 metres horizontal distance, the drift has dropped 3/5 metres or 0.6 metres, and this 0.6 metres would be added or subtracted to measurements taken at the

original grade line. The point thus located on the side wall towards the face would be marked with a peg inserted into the side wall. The procedure is repeated for another 3 metres and a

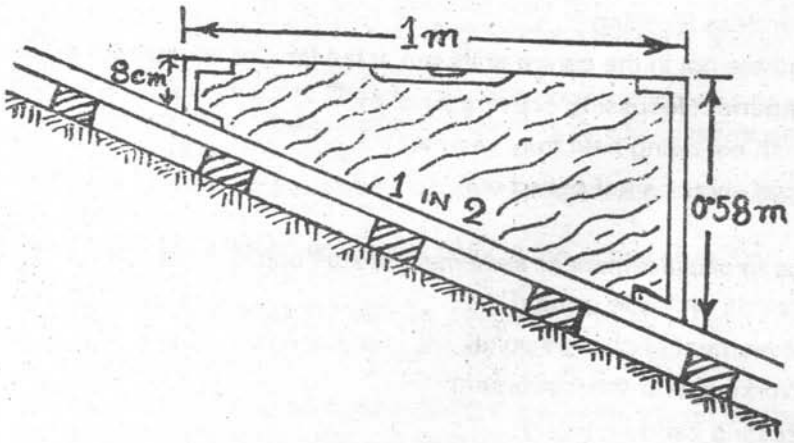


Setting out a Grade line

second spot on the correct grade established which is fixed with a peg. The three pegs are joined up using chalked cotton string and a line is drawn underneath the string as a reference to the workmen.

This precedure is repeated to extend the grade line at intervals of approximately 10 metres. Check levellings with reference to a previous Bench Mark are carried out using the-odolite and a staff to see that the drift is at its correct gradient.

A template made of wood with a long spirit-level fitted at its top is used to maintain the



Wooden template for 1 - in - 2 Gradient

gradient approximately. The inclined base of the template is placed on the rails of the drivage as it proceeds and the track is lifted or lowered until the bubble of the spirit-level comes to centre of its run. But periodical check must be provided by levelling the drivage with a theodolite to achieve the accuracy of the gradient.

ERRORS IN LEVELLING

CAUSES OF ERRORS : The various causes of errors in levelling are -

1. Instrumental errors.
2. Errors in manipulation.
3. Errors due to displacement or settlement of staff and level.
4. Errors due to natural causes.
5. Errors in reading.
6. Errors in booking.

1. *Instrumental errors may arise due to :*
   (a) Defect in construction of instrument and its bubble.
   (b) Imperfect adjustment.
   (c) Loose joints of tripod stands.
   (d) Inaccurate division of the staff.
   (e) Defective joints and foot plates of staff.

2. *Errors in manipulation may be due to :*
   (a) Careless levelling.
   (b) Bubble not in the centre of its run at the time of taking staff reading.
   (c) Imperfect focussing causing parallax.
   (d) Staff not being held truly vertical.
   (e) Staff sections not pulled out.

3. *Errors due to displacement or settlement of staff and level due to :*
   (a) Setting on loose ground.
   (b) Movement of change point.
   (c) Working near the instrument
   (d) Resting hand on tripod.

4. *Errors due to natural causes may arise due to :*
   (a) Effect of sun and wind and level when observations are made on a hot or stormy day.
   (b) Effect of cuvature of earth which increases staff reading.
   (c) Refraction which decreases staff reading.
   (d) Change of staff length with variation in temperature.

5. *Errors in reading due to :*
   (a) Sights too long.
   (b) Reading upwards instead of downwards.
   (c) Reading downwards during inverted staff reading.
   (d) Reading against stadia hairs.
   (e) Reading wrongly omitting zero from decimals or noting the whole number wrongly.

6. *Errors in Booking includes :*
   (a) Entering readings in wrong column.
   (b) Noting a reading with digits interchanged.
   (c) Entering a wrong distance.
   (d) Omitting an entry.

PRECAUTIONS TO ELIMINATE ERRORS IN LEVELLING : The following precautions should be taken to eliminate errors in levelling -

1. The instrument should be firmly set on solid ground : Its levelling screws should not be excessively tightened and each levelling screw must be in contact with the lower plate or bar.

2. A length of sight which may cause staff-gradations indistinct should be avoided.

3. Sight should not be long and the back and fore sights should be of equal length to eliminate errors due to instrument imperfections and errors due to cuvature and refraction.

4. The telescope should be accurately focussed and parallax eliminated.

5. The bubble must always be brought exactly to the centre of its run.

6. Errors from unstable supports can be avoided by using a heavy firm stand and repeating the levelling in the opposite direction and by taking the mean of the results.

7. Staff should be held firmly on solid ground. Crow's feet should be used to support the staff at the same height when it is turned to face the next set-up of the instrument.

8. Staff should be held in a truly vertical position. A staff level or a plumb line may be used by the staff holder to ensure the verticality of the staff.

9. If a slide of the staff is drawn out, it must be drawn out fully.

10. Foot of the staff should be kept free from dirt.

11. When velocity of wind exceeds 12-15 km per hour work must be kept in abeyance.

12. During heated atmosphere work should be proceeded with short sights only.

13. Care should be taken in reading and booking of staff readings.

LEVELLING OPERATIONS IN SOME SPECIAL CIRCUMSTANCES

Levelling with Broken Bubble Tube : If the bubble tube of your levelling instrument is broken due to carelessness of your assistant, levelling may be carried on when the edge of the object end of the telescope is carefully made vertical by a plumb line and the level is placed midway between the two pegs.

Levelling with Bubble not remaining Central : If the bubble of your dumpy level is not remaining central, the levelling operation is to be carried out by placing the level midway between two staff stations so that error of collimation is eliminated.

Levelling with Damaged Cross-hairs : If the cross-hairs of your dumpy level are accidentally broken while you have a short distance (150m to 200m) of approximately level underground road to cover to complete a check in levelling after a day's work, such mishap may cause delay in the completion of the work. In such cases the work would proceed in the usual manner except that for each sight to the staff two readings should be taken instead of one. The graduations on the staff cut by the upper and lower limits of the object glass as viewed through the eye-piece would be noted. The sum of these two readings divided by 2 is the true or correct reading, the reading which would be cut by the cross-hairs. To eliminate possible errors in collimation nearly equidstant back and fore shights would be taken.

EXERCISES

1. Define the following terms :
   (a) Line of collimation. (b) A level surface. (c) Parallax. (d) Change point (e) Back sight and Fore sight. (f) Collimation errors (g) Principle of reversal. (h) Bench Mark. (i) Differential levelling

2. (a) What is levelling and what is its purpose in connection with mining operations ?
   (b) What are the different types of levels ?

3. What is the difference between an ordinary level and a precise level ? What are the advantages claimed for a precise level ? **(A. M. I. E. 1985)**

4. Describe the salient features of a modern level and a precise level. Describe the merits and demerits of these features. Explain the principle of the parallel plate micrometer. **(A. M. I. E. 1984)**

5. Sketch and describe a Dumpy Level and explain its temporary adjustments. **(Survey Comp. 1956)**

6. What are the permanent adjustments of a Dumpy level ? How will you carry out the permanent adjustments of a Dumpy Level ?

7. Describe with sketches how the adjustments of a Dumpy Level are made before commencing an important survey. What precatuions should be observed to ensure accurate observation ? **(Survey Comp. 1964)**

8. What do you mean by Trigonometrical Levelling ? What are the advantages and disadvantages over the ordinary levelling with Dumpy Level ? **(Survey Comp. 1963)**

9. What is Reciprocal levelling ? What are the advantages of this system of levelling ?

10. Explain the meaning of curvature and refraction and their effect upon levelling operations. How would you eliminate the need for correcting their effect in your field work ?

11. State what is meant by "Precise Levelling" and how it differs from other systems of levelling. Describe a method of precise levelling overground with slopes not exceeding 1 in 20.

12. What is meant by the sensitivity of a spirit level ? How will you determine the sensitivity of the spirit-level of your Dumpy Level ?

13. What is subsidence surveying ? As a colliery surveyor what surveys will you carry out and what records will you maintain with regard to subsidence ? **(Survey Comp. 1985)**

14. It is desired to prepare a subsidence plan for an area (1km x 5km) of a mine. Describe the method of surveying and computation required if the mine is being worked by longwall retreating method, and the seam thickness is 2.5m. **(A. M. I. E. 1983)**

15. It is proposed to establish the level of an important underground bench mark with reference to a permanent bench mark on the surface. Describe in detail how you would proceed with the work, maintaining a high degree of accuracy. Assume the mine is served by vertical shafts.

16. Describe briefly how you would determine and thereafter lay down on plan, the contour lines of a surface area of 5m intervals :
    (i) Where the ground is comparatively flat, and
    (ii) Where the ground is fairly steep.

17. How will you carry on contouring in a hilly area ? Give precautions in the operation. **(A. M. I. E. 1985)**

18. Write short notes on :
    (a) National Datum - Mines. **(Survey Comp. 1981)**
    (b) Precise levelling. **(Survey Comp. 1983)**
    (c) Reciprocal levelling. **(A.M.I.E. 1983, 1984, 1985)**
    (d) Use of contour lines on mine plans. **(A.M.I.E. 1983, 1985)**
    (e) Subsidence survey. **(Survey Comp. 1984)**
    (f) Isopachyte line. **(Survey Comp. 1983, 1984)**
    (g) Grade contours. **(A.M.I.E. 1984)**
    (h) Parallel plate micrometer **(Survey Comp. 1984)**
    (i) Laser alignment. **(A.M.I.E) (Survey Comp. 1984)**
    (j) Correction for curvature in long distance levelling **(A.M.I.E. 1983)**

19. (a) Enumerate the various causes of errors in levelling by instruments.
    (b) State precautions that you will observe when using the level and staff so that the errors are reduced to a minimum. **(A. M. I. E. 1984)**

20. The reading taken on a staff 100m from the instrument with the bubble central was 1.872m. The bubble is then moved 5 divisions out of the centre, and the staff reading is observed to be 1.906m. Find the angular value of one division of the bubble and the radius of curvature of the bubble tube. The length of one division of the bubble is 2mm. **(Ans.** 14.03 seconds, 29.41m)

21. A level set up on extended line BA is in a position 70 metres from A and 100 metres from B reads 1.684 on a staff held at A and 2.122 on a staff held at B, the bubble having being carefully brought to the centre of its run before each reading. It is known that the reduced levels of the tops of the pegs A and B are 89.620 and 89.222 respectively. Find (a) the collimation error, and (b) the readings that would have been obtained had there been no collimation error. **(Ans.** (a) 4'34" upwards (b) A = 1.591 and B = 1.989m)

22. In a two peg test, the readings on two pegs A and B were 3.000m and 3.600m when the level was in between. The level was then set at C, 25m away from A, in line BA produced and the readings on A and B were 2.800 and 3.100m. The distance between A and B was 75m. Find the staff readings at A and B to give a horizontal line of sight.
    (Ans. At A = 2.900m, at B = 3.500m)

23. The following consecutive readings were taken with a level and a 4m staff on a continuously sloping ground at a common interval of 30m -
0.780, 1.585, 1.955, 2.430, 2.985, 3.480, 1.155, 1.960, 2.365, 3.640, 0.935, 1.045, 1.630 and 2.545.

The reduced level of first point was 180.750m. Rule out a page of level-book and enter the above readings. Calculate the reduced levels of all the points by rise and fall method and also the gradient of the line joining the first and last point.
(Ans. R.L. of 1st C. P. = 178.050m, R. L. of 2nd C. P = 175.565m, R.L of last point = 173.955m Gradient = 1 in 48.56 falling)

24. The following consecutive readings were taken with a level and 3m levelling staff on continuously sloping ground at a common interval of 20 metres -
0.602, 1.234, 1.860, 2.574, 0.238, 0.914, 1.936, 2.872, 0.568, 1.824, 2.722. The reduced level of the first point was 192.122. Rule out a page of a level field-book and enter the above readings. Calculate the reduced levels of the points, and also the gradient of the line joining the first and the last points.
(Ans. 192, 122, 191.490, 190.864, 190.150 (C.P), 181.474, 181.452, 187.516(C.P), 186.260, 185.362. Gradient 1 in 23.82 falling.)

25. The following series of readings of back sights and fore sights was taken in a fly levelling. The first reading was taken on a point of R. L. 100.00m. Draw a page of levelling field-book and enter the readings on it. Find the R.Ls of all points.
Apply Check : 1.235, 1.396, 2.345, 1.986, 2.148, 3.755, 0.325, 2.568, 1.465, 2.435, 1.356, 0.768, 1.985, 2.655. **(Ans.** R. L. of last point = 98.296)

26. It was required to ascertain elevations A and B. A line of levels was taken from A to B and then continued to a bench mark of elevation 42.434m. The observations are recorded below. Obtain the R. Ls of A and B.

B. S.	I. S.	F. S	R. L.	Remarks
1.306				A
0.485		2.560		
2.350		1.090		
	0.787			
1.633		0.283		B
2.876		0.990		
2.337		1.064		
		1.026	42.434	B.M

(Ans. R. L. of A = 38.860; R. L. of B = 38.564)

27. The staff readings for a survey work were as follows -
1.810, 2.110, 1.225, 1.455, 0.905, 2.435, 2.810, 2.675 and 1.765.
The level was shifted after the 4th and 7th reading. The first reading was taken on a bench mark of R. L. 50.000. Rule out a page of level book and enter the readings. Work out the R. Ls of all stations. If the staff were held inverted and readings on a ceiling from last instrument position was 3.500, find the R. L. of the ceiling. Work out the staff readings on top of 4 pegs at 20m intervals from the last station to give an upgrade of 1 to 100.
(Ans. R. L. of 1st C. P. = 50.355, R. L. of 2nd C. P. = 48.450. R. L. of end point = 48.760, R. L. of ceiling = 54.025, Staff readings on peg 1 = 1.765, peg 2=1.565, peg 3 = 1.365, and peg 4 = 1.165).)

28. Flying levels were taken from a colliery B.M. of R. L. 660.65 as for grading a haulage road of an incline, and the following staff readings were obtained -
Back sight : 3.63. 6.90, 4.02, 2.21, and 1.98.
Fore sight : 2.36, 4.85, 6.10, and 4.87.

From the last position of the instrument it is required to drive seven pegs at 50m interval with the top of pegs on a uniform gradient of 1 in 40 in favour of load. The first peg is to have a R. L. of 659.75m. Work out the staff readings required for setting the top of the pegs on given gradient.
(Ans. Peg 1 to Peg 7 staff readings are 1.46, 1.71, 3.96, 5.21, 6.46, 7.71 and 8.96)

29. A dumpy level was set up with its eye-piece over peg A. The height of the instrument above the peg was 1.31. The reading over another peg B was 0.900. The level was then removed and set up near peg B. The height of the instrument at B was 1.43. The reading on the staff at A was 1.84. Determine the actual R. L. on peg B, if R. L. on peg A was 100.00. **(Ans.** 100.41m)

30. A level was set up at a station O and the readings on the staff when held at A, 1.200m away from O, is 1.496 and when held at B, 800m away from it, is 2.855m. Find the true difference of level between A and B. **(Ans.** 1.443m)

31. Find the error of reading of a levelling staff if the observed reading is 3.986m and the point sighted at the staff 15cm off the vertical through the bottom. Calculate also the correct staff reading. **(Ans.** + 0.003m, 3.983m)

32. A 4m levelling staff was found to be 20cm out of plumb at one of its ends. If the reading on the staff is 3.100m, calculate the correct reading. **(Ans.** 3.096m)

33. The observed reading on a staff held at A was 3.695m. The staff was found to be 10cm off the vertical. Find the correct staff reading on A. **(Ans.** 3.694)
(Survey Comp. 1974)

CHAPTE1R - VIII

DIP AND FAULT PROBLEMS

★★★

If AB is the direction of full dip in the figure, then the dip along any line such as AC is an apparent dip. Let the full dip = α and the apparent dip = β

$$\text{Tangent } \alpha = \frac{AD}{BD}$$

$$\therefore \quad AD = BD \tan \alpha$$

Similarly, $AD = CD \tan \beta$

$$\therefore \quad CD \tan \beta = BD \tan \alpha$$

$$\therefore \quad \tan \beta = \frac{BD}{CD} \tan \alpha$$

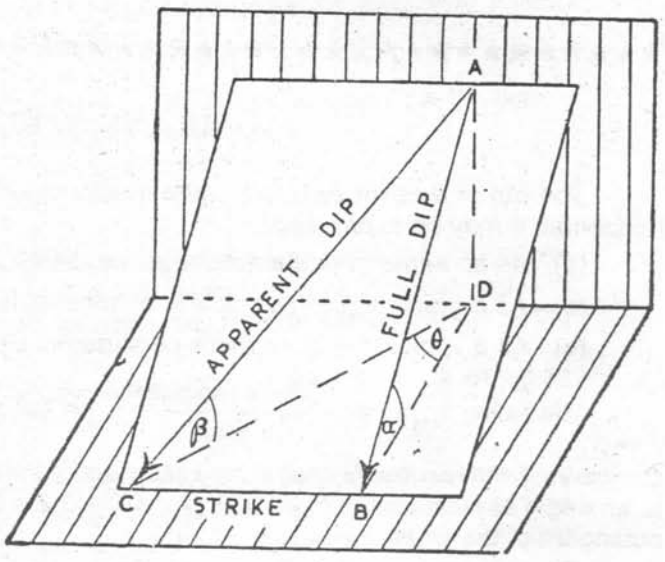
$$\therefore \quad \text{Cot } \beta = \frac{CD}{BD} \cot \alpha$$

In triangle BCD, angle B = 90°

$$\therefore \quad \frac{CD}{BD} = \text{Secant } \theta$$

$$\therefore \quad \textbf{Cot } \beta = \textbf{Cot } \alpha \cdot \textbf{secant } \theta$$

Hence **Cot apparent dip = Cot full dip x sec of the angle between apparent and full dips**



Relation between Apparent and Full dips

The above formula may also be written in various ways, as desired, to calculate any one unknown factor if the other two factors are given. Thus -

$$\text{Cot } \beta = \text{Cot } \alpha \quad \text{Sec } \theta \quad \text{or Tan } \beta = \text{Tan } \alpha \quad \text{Cos } \theta$$

Tan apparent dip = tan full dip x cos angle between full and apparent dips.

Example 1. The amount and direction of full dip is 1 in 2 due South. What is the gradient of a road driven on a bearing S40° W ?

Solution : We know -

Cot apparent dip = Cot full dip x sec of included angle.

= 2 x sec 40°

= 2.6

Hence the gradient of road is a fall of 1 in 2.6.

Example 2. The full dip of a seam is 1 in 4 due south. Calculate the apparent dips in the directions (a) S 30° E, (b) S 45° E, and (c) S 60° E.

Solution : Let α = apparent dip, θ = full dip, and ϕ = horizontal angle between two dips. Then $\tan \alpha = \tan \theta \cos \phi$.

(a) **Apparent dip towards S 30° E :**

$\tan \alpha = \tan \theta \cos 30^\circ = {}^1/_4 \times 0.866 = 0.2165$, and apparent dip α = **12°13'.**

$\cot \alpha = \cot \theta \sec 30^\circ = 4 \times 1.155 = 4.62$, and apparent dip α = **1 in 4.62**

(b) **Apparent dip towards S 45° E :**

$\tan \alpha = \tan \theta \cos 45^\circ = {}^1/_4 \times 0.7071 = 0.1768$, and apparent dip = **10°02'.**

$\cot \alpha = \cot \theta \sec 45^\circ = 4 \times 1.4142 = 5.66$, and apparent dip = **1 in 5.66**

(c) **Apparent dip towards S 60° E :**

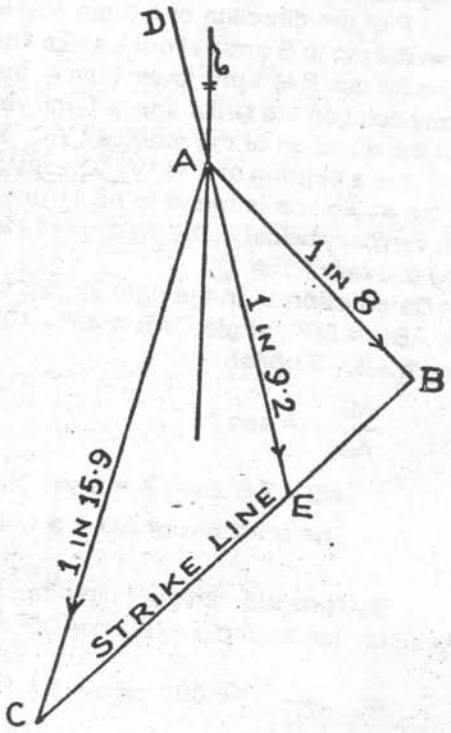
$\tan \alpha = \tan \theta \cos 60^\circ = {}^1/_4 \times 0.5 = 0.125$, and apparent dip = **7°08'.**

$\cot \alpha = \cot \theta \sec 60^\circ = 4 \times 2 = 8$, and apparent dip = **1 in 8.**

Problems on gradient may also be solved graphically which is sufficiently correct for many purposes and should always be used to check results obtained by trigonometry. The manner of solving these problems graphically depends on whether the gradients are stated in terms of *1 in N*, or as angles of inclination. The methods to be adopted are best shown by a number of examples.

Example 3. In a seam of coal the direction of full dip S 45° E, and the rate of dip is 1 in 8. Find the inclination of two slope heading, the azimuth of which are 195° and 345°

Solution : Figure represents a plan of the heading referred to and is constructed as follows-



Draw a line AB, in the direction of full dip, namely S 45° E in this case, and 8 units long, the units being of any convenient length. Through B draw a strike-line, BC, at right-angles to AB. Let B is 1 unit vertically lower than A. Then no matter in what direction a road is driven from A to meet the strike-line BC, it will fall 1 unit vertically in the horizontal distance N units, represented to scale by the length of the line depicting the road on the plan. This length, N units, will also represent the gradient of the road expressed as 1 vertical N horizontal.

Now draw AC in the direction S 15° W (the azimuth being 195°) and AD in the direction N 15° W (the azimuth being 345°). AC is the apparent dip being in the direction of dip, but the heading AD is the apparent rise being in opposite direction. DA is extended to meet the strike-line at E. Measure the lengths of AC and AE in the same units as AB.

By measurement, AC = 15.9 units long.
AE = 9.2 units long.

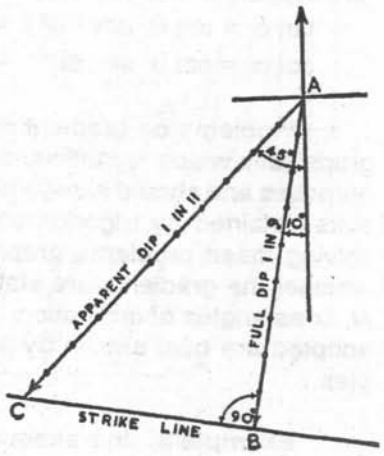
∴ **Apparent dip in the direction 195° azimuth = 1 in15.9 (= 3°36')**
and, apparent rise in the direction 345° azimuth = 1 in 9.2 (= 6°12')

Example 4. Levels are taken in a roadway advancing direct to the rise on a bearing N 10° E, and the gradient is found to be 1 in 9. What will be the gradient of another roadway which is being driven on an azimuth of 225° ? Your answer may be obtained by construction or by calculation.

Solution : Since the direction of full rise is N 10° E, the direction of full dip is S 10° W. The rate of full dip is 1 in 9.

The solution of the problem can be obtained by construction as shown in figure.

Plot the direction of full dip AB. Measure 9 units down the dip to B and set out a strike line at right angles to the full dip. B is 1 unit lower than A, therefore the level of any point on the strike-line is 1 unit vertically below A, Plot the direction of the roadway, AC, on an azimuth of 225° , i.e. a bearing of S 45° W. The distance from A to C on the strike-line is found to be 11 units, and as C is 1 unit vertically below A, the gradient of the proposed roadway is a fall of 1 in 11.



By Calculation : In the right-angled triangle ABC, angle ABC = 90° , angle CAB = 45° - 10° = 35° , and the length AB = 9 units.

$$\frac{AC}{AB} = \sec A$$

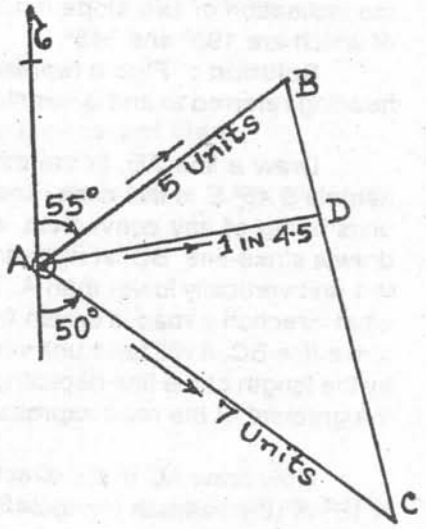
∴ AC = AB x sec A = 9 sec 35° = 11.

∴ **The gradient of AC is a fall of 1 in 11.**

By formula, tan app. dip = tan full dip x cos ϕ
or, tan app. dip = $^1/_9$ x cos 35° = 0.091

or, $\frac{1}{N} = 0.091$, when N = 11

Hence the **apparent dip along AC is 1 in 11.**



Example 5. A seam of uniform gradient dips at 1 in 7 in the direction of S 50° E and 1 in 5 in the direction of N 55° E. Determine, using a graphical method, the direction and rate of full dip of the seam. **(Survey comp. 1986)**

Solution : Figure represents a plan of the direction of dip referred to and is con-

structed as follows :

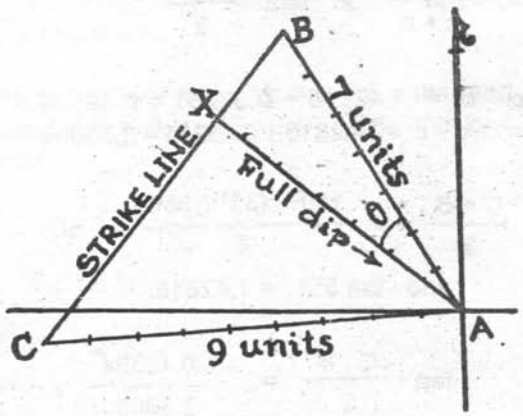
A line AB is drawn in the direction N 55° E and 5 units long. A line AC, 7 units long, is also drawn in the direction S 50° E. BC is strike line. A line AD perpendicular to BC is the line of full dip. AD is the direction of full dip of the seam = N 80° 15'E at 1 in 4.5.

Example 6. The following obsetvations were made in two roads AB and AC in a seam :

Road	Bearing	Inclination
AB	N 27°30' W	Rising 1 in 7
AC	S 85°36' W	Rising 1 in 9

Calculate the direction and rate of full dip of the seam.

Solution : As a check on the calculated result, first plot AB N 27° 30'W and 7 units long; AC S 85°36' W and 9 units long and join BC. BC is the strike line. Construct XA at right angles. The XA is the direction of full dip = S 50° E at 1 in 6.5 approximately.



Calculation : Angle BAC = 180° – (27°30' + 85°36') = 180° – 113°06' = 66°54'.

$$\text{Tan } \frac{B-C}{2} = \frac{b-C}{b+C} \cot \frac{A}{2}$$

$$= \frac{9-7}{9+7} \cot \frac{66°54'}{2} = \frac{2}{16} \text{ Cot } 33°27' \quad \therefore \quad \frac{B-C}{2} = 10°43'$$

or, $B - C = 21°26'$

and $B + C = 113°06'$

$2B = 134°32' \quad \therefore B = 67°16'$

$C = 113°06' - 67°16' = 45°50'$

Now, in triangle ABX, XA = 7 Sin 67°16' = 6.4561

Full dip of the seam = 1 in 6.4561

Angle BAX = 90° – 67°16' = 22°44'

Bearing of AX (rise) = N (27°30' + 22°44') W = N 50°14'W

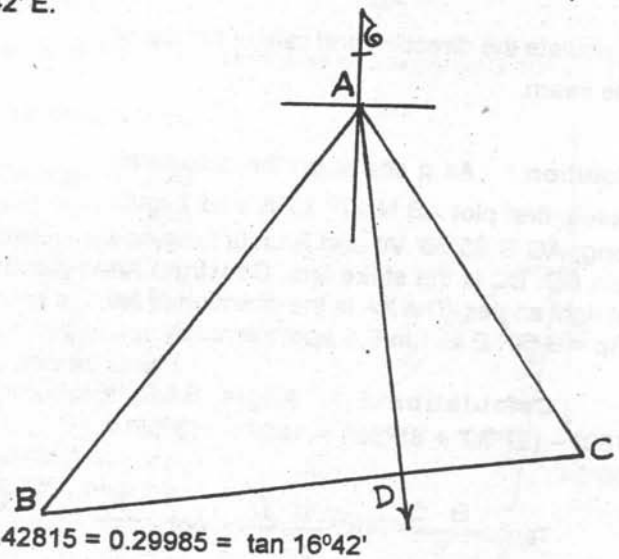
Direction of full dip = Bearing of line XA = S 50°14'E.

Example 7. The apparent dip of a lode, when measured in the direction S 35°W is 35°, and when measured in the direction S 35°E, is 47°. Calculate the amount and direction of dip of the lode. Check your result graphically.

Solution : AB in the figure represents apparent dip 35° towards S 35°W, and AC is the apparent dip 47° towards S 35°E.

Cot 35° = 1.42815, and Cot 47° = 0.93251.

AB is made 1.42815 units long and AC 0.93251 units. A line joining BC is the strike line. The line AD is drawn at right angle to BC to indicate the derection of true dip. Then true dip AD = 1 in 0.9, approximately towards S 16°42' E.



In triangle ABC, $\tan \frac{C - B}{2}$

$= \frac{c - b}{c + b} \times \tan \frac{C + B}{2}$

$c - b = 1.42815 - 0.93251 = 0.49564$

and, $c + b = 1.42815 + 0.93251 = 2.36066$

$\frac{C + B}{2} = \frac{180° - (35° + 35°)}{2} = 55°$

and $\tan 55° = 1.42815.$

$\tan \frac{C - B}{2} = \frac{0.49564}{2.36066} \times 1.42815 = 0.29985 = \tan 16°42'$

From $\frac{C + B}{2} = 55°$ and $\frac{C - B}{2} = 16°42'$, angle $C = 71°42'$.

Now in triangle ACD, angle $DAC = 90° - 71°42' = 18°18'$

∴ **Bearing of full dip = S (35° - 18°18') E = S 16°42' E**

Again, $\tan \text{true dip} = \frac{\tan \text{apparent dip}}{\cos \text{included angle}} = \frac{\tan 47°}{\cos 18°18'} = \frac{1.0723687}{0.9494254}$

$= 1.1294923 = \tan 48°29'$

∴ **Rate of full dip = 48°29' = 1 in 0.885.**

Example 8. Three boreholes A, B and C supplied the following informations of a coal seam :

Line	Bearing	Gradient
AB	S 40°W	1 in 5
AC	S 35°E	1 in 3

Calculate the direction and gradient of the true dip of the coal seam.

(L. Min. E, 1936)

Solution : To obtain a graphical result, draw AB S 40°W and 5 units long. Draw AC S35°E and 3 units long. Join BC and construct AD at right angles. Then AD is the direction of full dip = S 15°E at 1 in 2.8.

Calculation : The procedure involves solving the triangle ABC in order to find the angle at C; thence calculating the angle CAD and so deducting the bearing of AD from the known bearing of AC. Thereafter, the rate of dip may be found in the usual way.

In triangle ABC, we know the relative lengths of the two sides AB = c = 5 units, and AC = b = 3 units. We also know the included angle CAB = A = 40° + 35° = 75°. We can therefore find the value of the angle C from the formula -

$$\tan \frac{C - B}{2} = \frac{c - b}{c + b} \tan \frac{C + B}{2}$$

Now c - b = 5 - 3 = 2 units; and c + b = 5 + 3 = 8 units.

$$\text{also } \frac{C + B}{2} = \frac{180^\circ - 75^\circ}{2} = \frac{105^\circ}{2} = 52^\circ 30'.$$

$$\text{Hence } \tan \frac{C - B}{2} = \frac{2}{8} \times \tan 52^\circ 30' = \frac{1}{4} \times 1.3032 = 0.3258$$

$\frac{C - B}{2} = 18^\circ 03'$. But $\frac{C + B}{2} = 52^\circ 30'$ and thus by addition angle C = 70°33'

Now, in triangle ADC, angle CAD = 90° – 70°33' = 19°27'.

But AC = S 35°E and therefore AD = S (35°E – 19°27') **E = S 15°33'E** = bearing of full dip.

The rate of *full dip*, θ, may now be found from the formula : $\tan \alpha = \tan \theta \cos \phi$ for the rate of apparent dip α along AC is known (= 1 in 3) together with the horizontal angle ϕ = 19°27' contained between AC and the line of full dip AD. Thus. -

$$\tan \alpha = \tan \theta \cos \phi, \text{ Where } \tan \theta = \frac{\tan \alpha}{\cos \phi} = \frac{0.3333}{0.9429} = 0.3535.$$

Angle of full dip = **19°.28'**, = *1 in 2.8288*

Example 9. A coal seam dips due South at 20° from the horizontal. Find by calculation the bearings of roadways having a gradient of 1 in 5.

Solution : Let α = Apparent dip, θ = full dip, and ϕ horizontal angle between two dips.

Then $\tan \alpha = \tan \theta \cos \phi$

$$\cos \phi = \frac{\tan \alpha}{\tan \theta} = \frac{\cot \theta}{\cot \alpha} = \frac{\cot 20^\circ}{5} = \frac{2.7474}{5} = 0.5495$$

Or $\phi = 56^\circ 40'$

Hence the horizontal angle made by the roadway with the line of full dip is 56°40' and so the bearing of one road dipping at 1 in 5 is **S 56° 40' E.** It is evident, however, that a road at **S 56° 40' W** will also dip at 1 in 5, and that two other roads at **N 56° 40' W** and **N 56° 40'E** will also rise at 1 in 5, There are thus four possible directions in which the gradient is 1 in 5.

Example 10. The true dip of a seam is 14° in the direction of N 30° W. Calculate the dip of a road way, driven in the seam with a bearing of N 25° E.

Solution : Tan $\alpha = \tan \theta \cos \phi$ where, α = apparent dip, θ = full dip, and ϕ = horizontal angle between two dips.

$\tan \alpha = \tan 14^\circ \times \cos (30^\circ + 25^\circ)$

$= 0.24933 \times 0.57358 = 0.14301$

$\alpha = 8^\circ 08' =$ **apparent dip.**

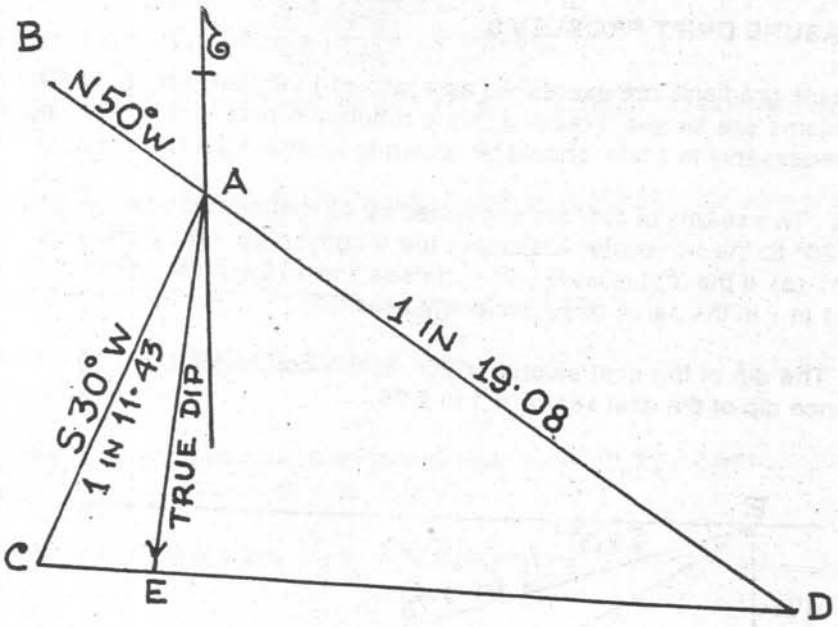
cot 8°08' = 7 and hence *the dip of the roadway in the direction of N 25° E = 8°08' or 1 in 7.*

Example 11. Two drives commence from a common starting point. One rises at an angle of 3° going N 50°W, the other dips 5° going S 30°W. What is the direction and grade of the true dip ?

Solution : AB in figure represents the drivage rising 3° towards N 50°W, AC dipping 5° towards S 30°W. The line BA produced to D, thus AD dips 3° towards S 50° E.

We know -

$\cot 3^\circ = 19.08$ and $\cot 5^\circ = 11.43$.



AC is made 11.43 units long, and AD 19.08 units. A line joining CD is the strike line. The angle CAD = 30° + 50° = 80°. The line AE is drawn at right angles to CD to indicate the amount and direction of true drip = 1 in 10 in a direction S 6° 30' W (approx).

Calculation : In the triangle ACD, the two sides are 11.43 and 19.08 units and the included angle CAD = 80°.

$\tan \frac{C - D}{2} = \frac{c - d}{c + d} \tan \frac{C + D}{2}$ where $c - d = 19.08 - 11.43 = 7.65$ units; $c + d$ $= 19.08 + 11.43 = 30.51$ units. $(C + D) \div 2 = (180^\circ - 80^\circ) \div 2 = 50^\circ$. $\tan 50^\circ = 1.1917536$.

$\therefore \tan \frac{C - D}{2} = \frac{7.65}{30.51} \times 1.1917536 = 0.2981172 = \tan 16°38'$.

$\therefore$ Angle C = 50° + 16°38'. = 66°38'.

Now in triangle CAE, angle CAE = 90° - 66°38' = 23°22'

$\therefore$ *Bearing of full dip* = S (30° - 23°22') W = **S 6°38' W.**

$$\text{Tan true dip} = \frac{\tan \alpha}{\cos \phi} = \frac{\tan 5^0}{\cos 23°22'} = \tan 5°27'$$

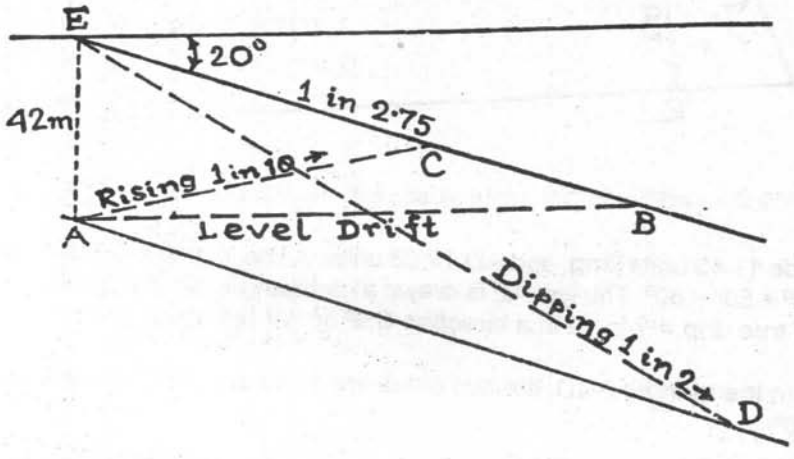
$\therefore$ *Rate of full dip* = 5°27' = **1 in 10.49.**

CROSS-MEASURE DRIFT PROBLEMS

Where the gradients are expressed as a ratio of 1 vertical to N horizontal, cross-measure drift problems are simple if tackled in the manner shown in the following examples. A sketch, not necessarily to scale, should be drawn to illustrate the conditions.

Example 12. Two seams of coal are separated by 42 metres (vertical) of strata which dip at an angle of 20° to the horizontal. Calculate the length of the cross-measure drift driven to connect them (a) if the drift is level, (b) if it rises 1 in 10 towards the dip of the seams and, (c) if it dips 1 in 2 in the same direction as the seams.

Solution : The dip ot the coal seams = 20° to the horizontal. Cot 20° = 2.74748 or 2.75 (approx), hence dip of the coal seams = 1 in 2.75.



(a) Level drift : The drift AB in the adjoining figure forms the base of a right-angled triangle ABE representing a gradient of 1 vertical in 2.75 horizontal, the perpendicular AE being 42m long.

$\therefore$ *Length of drift AB* = 2.75 x 42 = **115.50m.**

(b) Drift rising 1 in 10 : Here the drift AC and the seam EB are dipping in opposite direction. In all such cases, *where the drift and seams dip in opposite direction, the effective rate of approach is equal to the sum of the gradients.*

Thus the rate of approach $= \frac{1}{10} + \frac{1}{2.75} = \frac{51}{110}$

$\therefore$ AC approaches EB at the rate of 51 vertical in 110 horizontal, but AC and EB start 42 metres vertically apart (=EA).

$\therefore$ The drift meets the seam at a horizontal distance of $^{42}/_{51} \times 110 = 90.59$m. But we require the slope length of AC which reses at 1 in 10.

$\therefore$ *Length of drift AC* $= 90.59 \times \frac{\sqrt{101}}{10} =$ **91.04m.**

(c) Drift dipping 1 in 2 : In this case the drift ED and the seam AD both are dipping in the same direction.

Where the drift and seams dip in the same direction, the effective rate of approach is equal to the difference of the gradients.

Thus the rate of approach $= \frac{1}{2} - \frac{1}{2.75} = \frac{3}{22}$

Therefore ED is approaching AD by 3 units vertical in 22 horzontal. But they start 42 metres vertically apart.

$\therefore$ The drift meets the seam at a horizontal distance of $^{42}/_{3} \times 22$ metres = 308m. But we require the slope length of ED which dips at 1 in 2.

$\therefore$ *Length of drift ED* $= 308 \times \frac{\sqrt{5}}{2} =$ **344.35 metres.**

Example 13. Two seams of coal are 50 metres vertically apart and dip 1 vertical in 5 horizontal. Calculate in metres, the length of a cross-measure drift driven to connect them if it dips 1 in 3 in the same direction as the seams. What would be its length if it rises 1 in 10 towards the dip of the seams ? **(L. Min E. 1986)**

Solution : *Drift dipping 1 in 3.* In this case the drift and the seams dip in the same direction, and

Rate of approach $= {}^{1}/_{3} - {}^{1}/_{5} = {}^{2}/_{15}$

$\therefore$ Horizontal length of drift $= 50 \times {}^{15}/_{2} = 375$ metres.

But the drift rises at 1 in 3

$\therefore$ *Length of drift* $= 375 \times \frac{\sqrt{10}}{2} =$ **395.285 metres**

Drift rising 1 in 10 : In this case, the drift and the seams dip in opposite directions, and

Rate of approach = $^1/_5 + {}^1/_{10} = {}^3/_{10}$

∴ Horizontal length of drift = $50 \times {}^{10}/_3 = {}^{500}/_3$ metres.

∴ *Length of drift* = $\frac{500}{3} \times \frac{\sqrt{101}}{10}$ = **167.488 metres.**

Example 14. The full dip of a seam is 1 in 6 in direction S 40° E. A cross-measure drift rising at 1 in 7 bearing S 20°W is to be driven to intersect another seam which lies 40 metres vertically above. Calculate the length of the drift from floor to floor of the seams.

Solution : The rate and direction of dip of both seams are considered to be same.

The drift is rising 1 in 7 in a direction S 20°W.

∴ *The azimuth of drift* = (180° + 20°) = 200°

The seam is dipping at 1 in 6 in direction S 40°E.

∴ *The azimuth of full dip of seam* = (180° - 40°) = 140°

Now, *apparent dip of the top seam towards the direction of drift* -

Cot apparent dip = Cot full dip x Sec (angle between)

= 6 x sec (200° - 140°) = 6 sec 60° = 12

Hence apparent dip of seams = 1 in 12

Parting between two seams = 40 metres.

Since the drift at 1 in 7 and the seam at 1 in 12 (apparent dip) are approaching each other with dips in opposite directions -

Rate approach = $^1/_7 + {}^1/_{12}$ = $^{19}/_{84}$

Horizontal length of drift = $40 \times {}^{84}/_{19}$ = 176.84m

Slope length of drift = $176.84 \times \frac{\sqrt{50}}{7}$ = 178.63m

∴ The length of drift is 178.63 metres.

Example 15. A lode is dipping due North at an angle of 85° to the horizontal. From the outcrop the surface dips due North at 34° to the horizontal. A level drift is commenced to run due South from a point on the hill side 300 metres below the outcrop of the lode measured on the surface. What will be the length of the drift and at what distance on the lode will it reach the lode below its outcrop ? If a vertical shaft were to be sunk from the outcrop of the lode, what would be its depth if the bottom of it were connected with a level drift to intersect the lode at the same point which is reached by the drift from the surface ?

Solution : Let AB is the lode, CB is the level drift and AD is the proposed shaft. AC = 300 metres.

$\angle DAB = 90^0 - 85^0 = 5^0$ $\quad \angle DAC = 90^0 - 34^0 = 56^0$

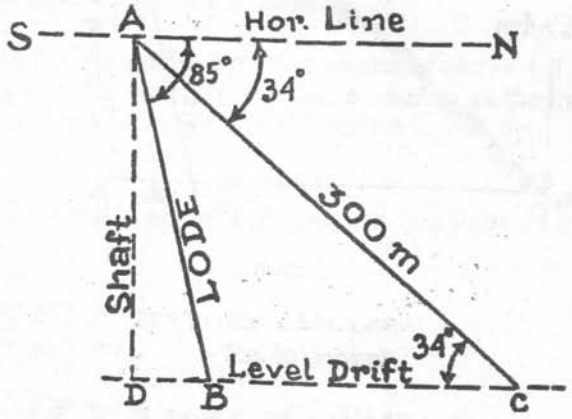
$\angle BAC = 85^0 - 34^0 = 51^0$ $\quad \angle ABC = 180^0 - (51 + 34^0) = 95^0$

Now in triangle BAC,

$$\frac{BC}{\text{Sin BAC}} = \frac{AC}{\text{Sin ABC}} \qquad \therefore \quad BC = \frac{300 \times \text{Sin } 51^0}{\text{Sin } 95^0} = 234.23\text{m}$$

Again, $\dfrac{AD}{\text{Sin } 34^0} = \dfrac{AD}{\text{Sin } 90^0}$

$$\therefore \quad AD = \frac{300 \times \text{Sin } 34^0}{\text{Sin } 90^0} = 167.76\text{m.}$$



∴ *Length of level drift BC*
= 234.03 metres.

Depth of shaft, AD
= 167.76 metres.

The distance from the outcrop to level drift along plane of lode,

$$AB = \frac{AD}{\text{Sin } 85^0} = \frac{AD}{\text{Sin } 85^0}$$

= 168.40 metres.

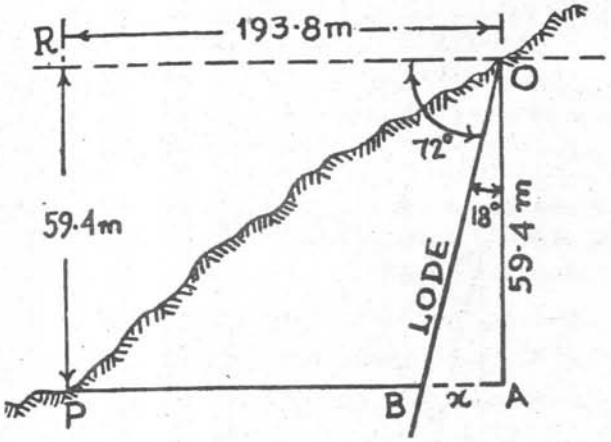
Example 16. O is a point on the outcrop of a lode which strikes due North and South and dips to the West at an angle of 72°. P is a point at the base of a hill from which it is proposed to drive an adit to cut the lode. P lies N 80°W of O, and at a reduced horizontal distance of 193.8m. O is 59.4m higher than P. Find the shortest length and the bearing of a level drift driven from P at which you would expect to cut the lode.

Solution : In the adjoining figure (section), OB is the lode, PB is the adit meeting the lode at B.

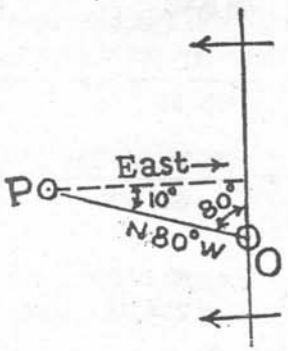
In triangle OAB, angle BOA = $90^0 - 72^0 = 18^0$, OA = 59.4 metres and let BA = x metres. Then

$$\frac{X}{OA} = \tan 18^0 \qquad \therefore \quad x = OA \tan 18^0 = 59.4 \times \tan 18^0 = 19.30\text{m}$$

∴ Length of adit, in direction S 80° E (opposite to line OP) to cut the lode = PA – BA = OR – x = 193.80 – 19.30 = 174.50 metres. This, however, is not the shortest distance, but is the hypotenuse of right-angled triangle, one of whose side points due East (just to the opposite direction of the dip of the lode) – the shortest distance of length of adit.



Section



Plan

∴ *Shortest length of adit* = 174.50 x Cos 10° = **171.85m.**
Bearing of adit = Opposite the dip of lode = **Due East.**

Example 17. The coal seams dip S 10° E at 1 in 5 and are 45 metres apart vertically. Calculate a level cross-measure drift driven in the direction of S 20° W.

Solution : To find out the apparent dip in the direction S 20° W.

Tan apparent dip = tan full dip x cos of included angle.

Here the included angle between the true dip and apparent dip is (10° + 20°) = 30°.

Hence

Tan apparent dip = 1/5 x cos 30° = 0.173205
∴ Apparent dip = 9°49'35" = 1 in 5.773

The drift forms the base of a right-angled trangle. Its hypotenuse represents the gradient of 1 in 5.773 and the perpendicular = 45 metres.

∴ **Length of level drift** = 45 x 5.773 = **259.785 metres.**

Example 18. A vertical borehole passes through two coal seams and indicates that they are 45m apart. The true dip of the seams is 20 degrees. A rising tunnel is driven from the lower seam in the direction of true dip. The angle of inclination of the tunnel is 14 degrees. Calculate the plan length and true length of the tunnel.

Solution : Let x metres be the vertical difference in height between the two ends of the tunnel.

x = Plan length x tan 14°, and

$45 - x$ = Plan length x tan 20°.

or, $x = 45 -$ Plan length x tan 20°.

∴ Plan length x tan 14° = 45 − Plan length x tan 20°

or, Plan length (tan 14° + tan 20°) = 45

$$\textbf{Plan length} = \frac{45}{\tan 14^\circ + \tan 20^\circ} = \textbf{73.374 metres}$$

True length = Plan length x sec 14°

= 73.374 x Sec 14° = **75.620 metres.**

Example 19. A dipping tunnel is to be driven from a seam 85 metres vertically above it. The angle of dip of the tunnel is to be 10 degrees and it is driven on the same bearing as the true dip of the seams . True dip is 25 degrees. Find the true length of the tunnel and also the plan length.

Solution : Let x metres be the vertical difference in height between the two ends of the tunnel.

Then $x =$ Plan length x tan 10°

and $85 + x =$ Plan length x tan 25°

Therefore $x =$ Plan length x tan 25° - 85

equating, Plan length x tan 10° = Plan length x tan 25° − 85

or, Plan length (tan 25° − tan 10°) = 85

$$\therefore \textbf{Plan length} = \frac{85}{\tan 25^\circ + \tan 10^\circ} = \textbf{293.123 metres}$$

True length = 293.123 x Sec 10° = **297.645 metres.**

Example 20. Determine the length of a cross-measure drift rising at 1 in 4 in a direction S 40° E to connect two parallel seams which dip at 1 in 6 in a direction S 10° W and are separated by 45m of straum.

Solution : A dip of 1 in 6 = 9° 28'.

Angle BAC = 90° − 9° 28' = 80° 32'. The stratigraphical distancce between two seams, BC = 45 metres.

$$\text{Vertical distance, AB} = \frac{\text{CB Sin } 90^\circ}{\text{Sin } 80^\circ 32'} = \frac{45 \times 1}{0.9864} = 45.62 \text{ metres}$$

Again, Q. B. S 40° E = 180° - 40° = 140° Azimuth

Q. B. S 10° W = 180° + 10° = 190° Azimuth

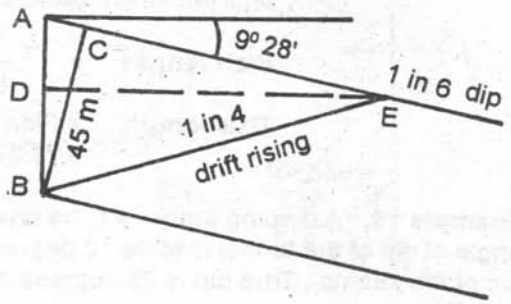
Dip of seams in direction of the drift is the apparent dip, and we know -
tan apparent dip = tan full dip x Cos angle between.

$\therefore$ tan apparent dip = $^1/_6$ Cos (190° – 140°) = 0.1071312

Therefore dip of seams in direction of drift = 1 in 9.334

Rate of approach of drift and seam

$$= \frac{1}{4} + \frac{1}{9.337} = \frac{13.334}{37.336}$$



Plan length of drift

$$DE = 45.62 \times \frac{37.336}{13.334} = 127.738\text{m}$$

Actual length of drift

$$= 127.738 \times \frac{\sqrt{17}}{4} = \textbf{131.67 metres.}$$

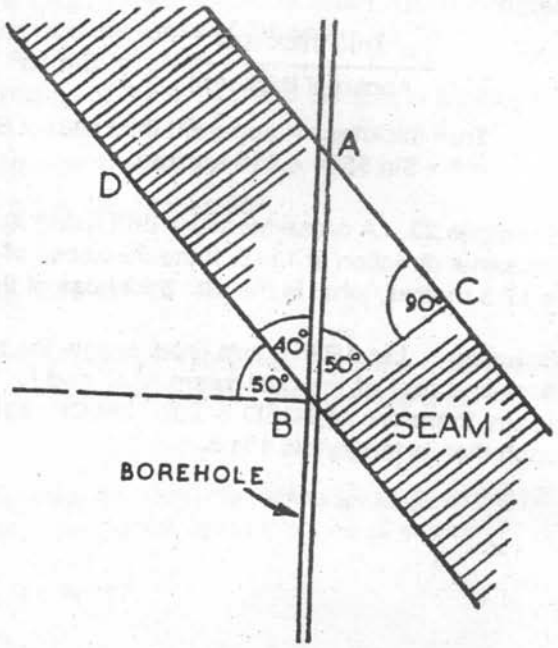
TRUE OR STRATIGRAPHICAL THICKNESS OF A SEAM

The stratigraphical thickness of an inclined seam is measured at right angles to the dip of the seam. When a vertical borehole is passed through any bed, the vertical thickness of the bed can be ascertained from the core. This is apparent thickness of the bed and the true or stratigraphical thickness may be determined if the dip of the bed is known.

Again, it is not unusual to find, in the course of putting down a borehole, that the boring tool becomes deflected from the vertical. By borehole surveying, the direction and amount of deviation at any point in the borehole may be determined, and consequently the true thickness of any bed passed through can be calculated if the apparent thickness is known.

Example 21. A vertical borehole passes through a seam of coal which is known to dip at 50°. If the apparent thickness of the seam as ascertained from the core is 8.6 metres, what is the true or stratigraphical thickness of the seam ?

Solution : In the adjoining figure, let AB represent that part of the vertical borehole which cuts the seam of coal.



Since the dip of DB = 50°, and the angle between the horizontal and vertical planes = 90°, it follows that angle ABD = 90° – 50° = 40°.

Since the stratigraphical thickness BC is measured at right angles to the dip DB, the angle ABC = 90° – 40° = 50°.

In the right-angled traingle ABC, $\frac{BC}{AB}$ = Cos ABC.

∴ BC = AB cos ABC
= 8.6 cos 50° = 5.53 metres.

∴ **True thickness of the seam = 5.53 metres.**

Example 22. A cross measure drift rising at an inclination of 20° passes through a seam of coal dipping in the oppostie direction at 35°. The thickness of the coal as measured along the drift is 6 metres. What is the true thickness of the seam ?

Solution : In fighure, the angle BAE is an exterior angle of the triangle ADE.

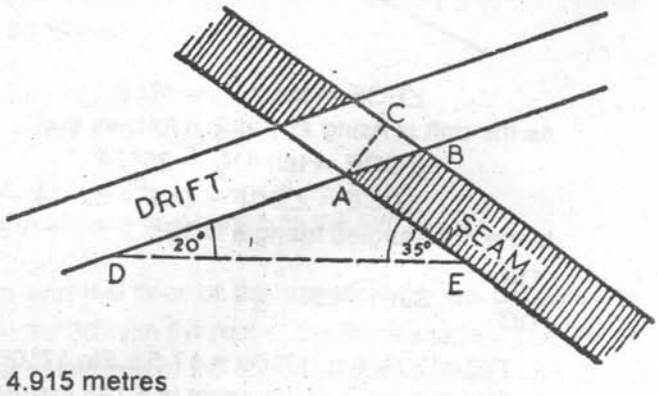
∴ ∠ BAE = ∠ ADE + ∠ DEA
= 20° + 32° = 55°

But ∠ CAE = 90°

∴ ∠ CAB = 90° - 55° = 35°

In the right angled triangle ABC,

AB = 6 metres,



∠CAB = 35°

$\frac{AC}{AB}$ = cos CAB

∴ AC = AB x cos CAB = 6 x cos 35° = 4.915 metres

∴ **Thickness of the seam = 4.915 metres.**

By Formula : Since the roof and floor of seam are parallel, the drift AB makes an angle of 55°

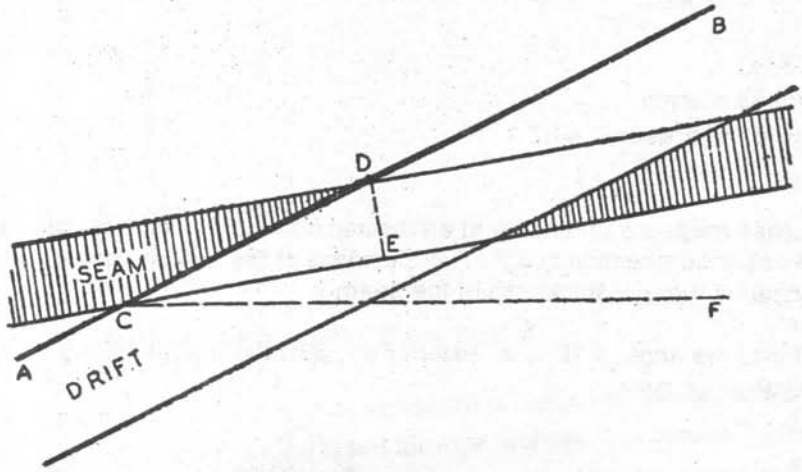
(∠ BAE) with the floor of the seam. Line of true thickness AC is perpendicular to the floor of seam.

$$\therefore \quad \frac{\text{True thickness}}{\text{Apparent thickness}} = \text{Sin } 55^0$$

True thickness = apparent thickness x Sin 55°
= 6 x Sin 55° = **4.915 metres.**

Example 23. A cross-measure drift rising at 1 in 2 passes through a seam of coal rising in the same direction at 1 in 6. If the thickness of the coal as measured along the roof of the drift is 17.5 metres, what is the true thickness of the seam ?

Solution : Let AB in figure reperesents the roof of the cross-measure drift, intersecting the floor and roof of the coal seam at C and D. Let DE be the stratigraphical thickness of the seam, therefore and CED = 90° . Let CF represents a horizontal line, then the angle ECF is such that its tangent is 1 in 6.



$\therefore \quad \angle ECF = \tan^{-1} 1/6 = 9^0 28'$

As the drift is rising at 1 in 2, it follows that :

$\angle DCE = \tan^{-1} 1/2 = 26^0 34'$

$\therefore \quad \angle DCE = \angle DCF - ECF = 26^0 34' - 9^0 28' = 17^0 06'$

In the right-angled triangle DCE,

$$\frac{DE}{DC} = \text{Sin } 17^0 06'$$

$\therefore$ DE = DC x Sin 17° 06' = 17.5 x Sin 17° 06' = 5.146 metres.

= *True thickness of the seam is 5.146 metres.*

Example 24. The dip of the strata at a colliery is 18° in a direction due South. A coal seam is passed through in a borehole and has a recorded thickness of 3.25 metres, but the borehole

is found to be deflected 10° from the vertical where it passes through the seam. The borehole is deflected in the direction S 30° E. Calculate the true thickness of the seam.

Solution : Apparent dip of the seam in the direction of the borehole S 30° E is to be calculated :

Tan apparent dip = Tan full dip x Cos angle between.

= tan 18° x (Cos 30° – 0°) = tan 18° cos 30°

= 0.2814

Hence, apparent dip towards the direction S 30° E = 15° 43'

The borehole makes an angle (90° – 10°) = 80° with horizontal, and the coal seam makes an angle 15° 43' with the horizontal. Hence the angle between the borehole and roof of the coal seam = 80° – 15° 43' = 64° 17'.

Since the roof and floor of the seam are parallel, the borehole makes an angle of 64° 17' with the floor of the seam. Line of true thickness is perpendicular to the floor of seam.

∴ True thickness = Apparent thickness x Sin 64° 17'

= 3.25 x 0.9009508

= **2.928 metres.**

Example 25. There are two coal seams each 2.4m thick dipping at 1 in 15 due S 35° W. An inclined upward borehole is drilled from the lower seam to the upper seam to dewater the latter and also to prove the parting between the two seams. The direction of borehole is S 55° W and it makes an angle of 55° to the vertical. If the borehole touches the floor of the upper seam at 22.5 metres, calculate the parting between the two seams.

Solution : Dip of seam is 1 in 15 in direction S 35° W. Direction of borehole S 55° W. Hence the apparent dip of seam in direction of borehole -

tan apparent dip = tan full dip x cos angle between.

= $^1/_{15}$ cos (55° – 35°) = $^1/_{15}$ cos 20°

Apparent dip = $\tan^{-1} {}^1/_{15}$ cos 20° = 3° 36'

The borehole is inclined at 55° to the vertical and connects the two seams.

Since the roof of the lower seam and the floor of the upper seam are parallel, the borehole makes an angle of 35° + 3° 36' = 38° 36' with the roof of the lower section. The line of true thickness of parting is perpendicular to the roof of the lower section. Hence -

The Parting between two seams = Apprarent thickness x Sin 38° 36'

= Length of borehole x Sin 38° 36'

= 22.5 x Sin 38° 36' = **14.04m**

DETERMINATION OF RATE AND DIRECTION OF FULL DIP OF A SEAM AFTER SINKING

The rate and direction of true dip of a coal seam is generally ascertained beforehand by putting boreholes as due consedetaion of the same is made in precise location of the position of headgear, winding engine, etc. before starting of sinking operations. After completion of sinking operations upto the seam when coal is exposed the rate and direction of full dip may be ascertained in the follwing manner -

Equipments required : Straight-edge 2m in length, spirit level, levelling staff, Dial, Plumb bob, cotton thread, chalk dust, etc.

Procedure : The straight-edge is laid across the line of dip towards the rise side on the floor of the coal seam in the shaft bottom, and by means of the spirit-level placed over it the exact line of strike AB is marked on floor by a chalked cotton thread. The line is again checked by the straight-edge whether it is on the same level.

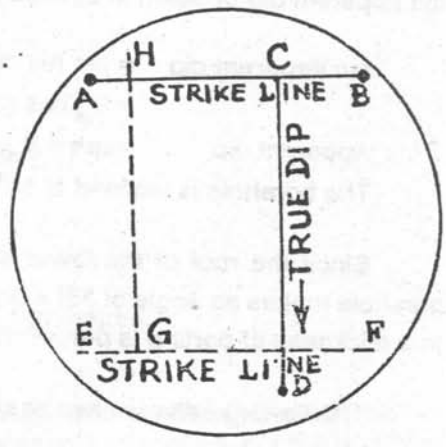
As we know that the full dip is always at right angles to the strike line, the dial is set up at C on the line AB and an angle BCD = 90° is set out. CD is the line of true dip which is marked off on the floor. The test should be repeated with the straight-edge placed on the dip side of the diameter of the space to mark the strike line EF and with the dial at G another line GH is marked off - at right angle to the strike line EF. Both the lines CD and HG are extended as far as the clear space permits.

The bearing of the lines CD and HG are found out by means of the dial. If the two bearings agree all is well; if not, their mean must be taken which is the direction of full dip which is noted.

To find the amount of dip the straight-edge, with the spirit level placed over it, is laid on the line of dip CD and the dip side end is raised some distance off the floor to bring the bubble to its central position indicating the straight-edge to a correct horizontal position. The difference of level is measured by means of a levelling staff held vertically which is checked by a plumb bob. Let this is 0.400 metres and the straght-edge being 2 metres long, the full dip of the coal seam is 1 in 5.

This test should be repeated at different points on the space, always placing the straight-edge along lines parallel with the full dip and the mean of several readings should be taken as the correct amount.

Finally a correlation survey between the surface base line of known bearing and the



Rate and direction of full dip

underground line of dip is made by precise magnetic correlation or by suspending two plumb wires in the shaft to find out the direction of full dip very precisely.

Example 26. A drift is to be driven from A, the co-ordinates of which are S 2763 and W 2150 in lower seam to B in upper seam, the co-ordinates of which are N 130 and E 217. Vertical distance between the seams is 60m. True dip of seam is 7° in a direction N 35° W. Calculate the bearing, inclination, plan length and true length of the drift. **(Survey Comp. 1978)**

Solution : Co-ordinates of A = − 2763, − 2150

Co-ordinates of B = + 130, + 217

To find the bearing of drift :

$$\tan(\text{bearing of AB}) = \frac{-2150-(+217)}{-2763-(+130)} = \frac{-2367}{-2893}$$

therefore bearing of AB = 39° 17' NE.

Plan length of drift $= \sqrt{2367^2 + 2893^2} =$ **3738m.**

True dip of seams = 7° in direction N 35° W, therefore, to find the dip of the seams in direction N 39° 17' E.

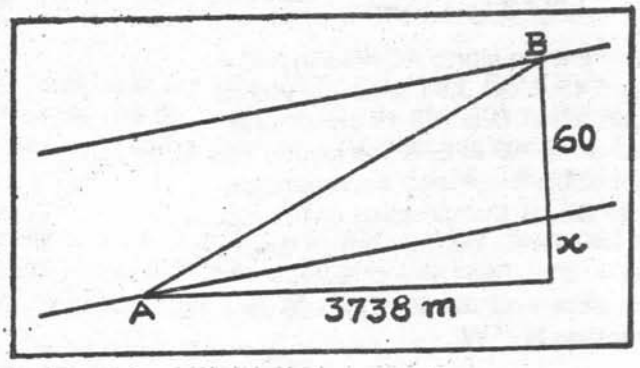
tan (apparent dip) = tan true dip x cos (included angle)

= tan 7° x cos (35° + 39°17')

= tan 7° x cos 74°17'

= 0.0332599

therefore, apparent dip = 1°54'



From the figure : X = 3738 x tan 1° 54' = 124m.

Difference of level between A and B = 124 + 60 = 184m

∴ **True length of drift** $= \sqrt{3738^2 + 184^2} =$ **3742.6m.**

Angle of inclination of drift :

tan (inclination) = $^{184}/_{3738}$ = 0.0492241 = tan 2° 49'

∴ **Angle of inclination = 2° 49'**

DIRECTION AND AMOUNT OF DIP FROM BOREHOLES

Given the the depths at which a seam of coal is encountered in three boreholes which are not in the same straight line, and whose positions and heights above some datum are known, it is possible to determine the dip of the seam in any direction, and also the probable

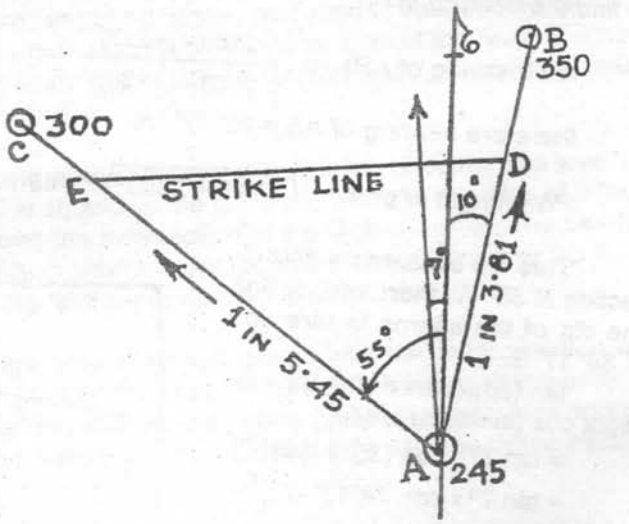
depth at which the seam would be reached by a shaft at any other point in the neighbourhood of the borehole. Problems of this type are usually solved by deducing the two apparent dips of the seam from the data given.

Example 27. Three boreholes A, B and C are put down to prove a coal seam. The depths at A, B and C are 245m, 350m, 300m respectively. B is 400m from A in a direction N 10°E, and C is 300m from A in a direction N55°W. Find, by plotting, the direction and amount of full dip of the seam. Assume the surface to be level. **(Survey Comp. 1980)**

Solution : On plotting the positions of the boreholes A, B and C and marking their depths in metres, it is seen that the seam dips along AB and AC. The fall from A to B is 350 - 245 = 105 metres and from A to C is 300 - 245 = 55 metres.

Apparent dip along AB = 105 in 400 = 1 in 3.81

Apparent dip along AC = 55 in 300 = 1 in 5.45. Now 3.81 units measured along AB to D and 5.45 units measured along AC to E. A line joining DE is the strike line. Now AF, perpendicular to DE, is the direction of full dip. AF = 3.65 units and angle NAF = 7°
Therefore Full dip is 1 in 3.65 in a direction N 7° W.



Example 28. Three bore-holes A,B and C have been put down to a coal seam. B is due North of A 1250m and C is N 60°W 800m from A. The surface levels of boreholes are the same. The depth of A = 750m, B = 1300m, and C = 950m. Calculate the rate and direction of full dip of the seam. **(Survey Comp. 1979)**

Solution : The positions of three boreholes A, B and C are roughly plotted and the dips along AB and AC are worked out from the given depths of the seam.

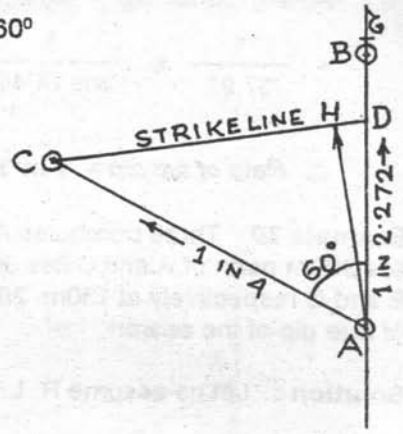
Dip AB = (1300 – 750) in 1550 = 1 in 2.272

Dip AC = (950 – 750) in 800 = 1 in 4

AC = 4 units, and AD (along AB) = 2.272 units. CD is the strike line and AH, perpendicular to CD, is the full dip.

In triangle CAD, $\tan \frac{D-C}{2} = \frac{d-c}{d+c} \tan \frac{D+C}{2}$

Where d = 4 and c = 2.272; $\frac{D + C}{2} = \frac{180^\circ - 60^\circ}{2} = 60^\circ$



$\therefore \tan \frac{D - C}{2} = \frac{4 - 2.272}{4 + 2.272} \times \tan 60^\circ$

$= \frac{1.728}{6.272} \times 1.732 = \tan 25^\circ 30'$

Angle D $= 25^\circ 30' + 60^\circ = 85^\circ 30'$

Angle HAD $= 90^\circ - 85^\circ 30' = 4^\circ 30'$

$\therefore$ *Bearing of full dip* = **N 4° 30'W**

Again, tan full dip = tan apparent dip ÷ cos (angle between)

$= \frac{1}{2.272} \div \cos 4^\circ 30' = \frac{1}{2.265}$

$\therefore$ *Rate of full dip* = **1 in 2.265 = 23° 49'.**

Example 29. Three boreholes A, B and C have been put down in a coal seam. B is 1200m due north from A and C is 750m N 65° W from A. The depth of A is 73.4m, of B 105m and of C 90m. The R. L. at the top of all the boreholes is the same. Calculate the direction and rate of full dip of the seam. **(A. M. I. E. 1983)**

Solution : First plot the positions of the three bores A, B and C, and work out the dips along AB and AC from the given depths to the seam. Figure is similar to the previous example (25).

Dip AB = (105 – 73.4) in 1200 = 31.6 in 1200 = 1 in 37.97

Dip AC = (90 – 73.4) in 750 = 16.6 in 750 = 1 in 45.18.

Measure AD = 38 units, and AC = 45 units. Join CD = the strike line and AH, perpendicular to CD, is full dip. Then *rate of full dip* = **1 in 34.5, N 25° W**

Calculation : In triangle CAD, $\tan \frac{D - C}{2} = \frac{d - c}{d + c} \tan \frac{D + C}{2}$

Here, d = 45.18; C = 37.97; $\frac{D + C}{2} = \frac{180^\circ - 65^\circ}{2} = 57^\circ 30'$

$\therefore \tan \frac{D - C}{2} = \frac{45.18 - 37.97}{45.18 + 37.97} \tan 57^\circ 30' = \frac{7.21}{83.15} \times 1.5698 = 0.1361 = \tan 7^\circ 45'$

Angle $D = 7^\circ 45' + 57^\circ 30' = 65^\circ 15'$

Now in triangle HAD, angle at $A = 90^\circ - 65^\circ 15' = 24^\circ 45'$

$\therefore$ Bearing of full dip = **N 24° 45' W.**

Agian, tan full dip = tan apparent dip ÷ cos 24° 45'

$$= \frac{1}{37.97} \times \frac{1}{\cos 24°45'} = \frac{1}{34.48}$$

∴ *Rate of full dip* = **1 in 34.48 = 1° 40'**

Example 30. Three boreholes A, B and C were put in a virgin area to prove a coal seam. B lies 300m north of A and C lies 360m east of A. The coal seam was touched by boreholes A, B and C respectively at 130m, 205m and 170m. Find by calculation the direction and amount of true dip of the seam. **(Survey Comp. 1977)**

Solution : Let us assume R. L. at surface to be 1000 metres.

	R.L. at bottom of seam
Depth of Borehole at A = 130m	870m
Depth of Borehole at B = 205m	795m
Depth of Borehole at C = 170m	830m

Let D be the point in line AB where the seam is met at the same level as at C, i.e. 830m R.L.

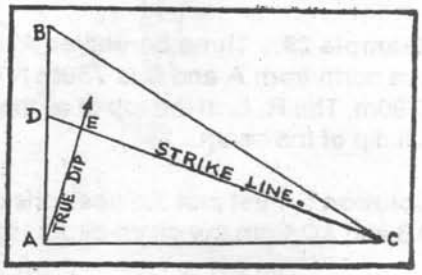
Now, fall from A to B = 870 – 795 = 75m

fall from A to D = 870 – 830 = 40m

Hence $\frac{AB}{AD} = \frac{\text{Fall along A to B}}{\text{Fall along A to D}}$

$$\therefore AD = \frac{AB \times \text{Fall along A to D}}{\text{Fall along A to B}}$$

$= 300 \times {}^{40}/_{75} = 160m$



Since D is at same level as C, DC represents the strike line and AE, the line drawn perpendicular to DC represents the true dip direction. In triangle ACD,

$$\angle ABD = \tan^{-1}\frac{AD}{AC} = \tan^{-1}\frac{160}{360} = \tan^{-1}\frac{4}{9} = 23°58'$$

In triangle ACE, AE = AC sin ACD = 360 sin 23°58' = 146.21m

Now fall from A to E = 870 – 830 = 40m

Hence *Amount of dip* = $\frac{40}{146.21}$ = **1 in 3.655**

Now, ∠ DAE = ∠ECA = 23°58'

∴ *Direction of true dip* = **N 23°58' E.**

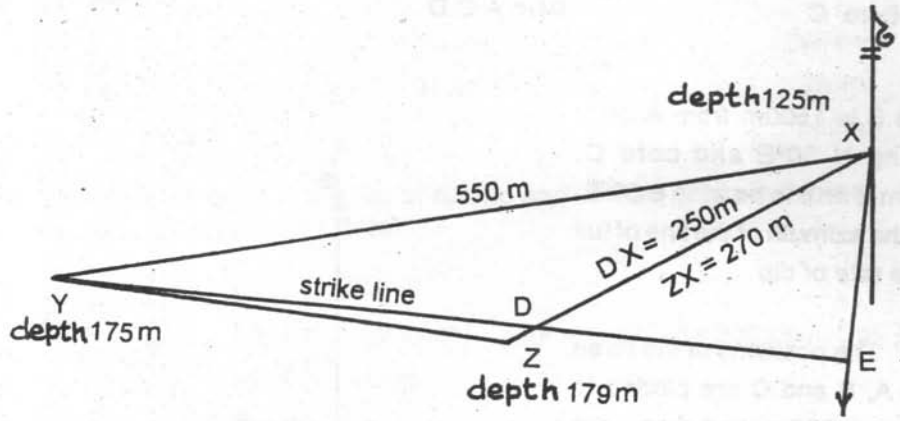
Example 31. Three boreholes X, Y and Z have been put down to a coal seam; Y is S 10°W and Z is S 52°W from X. Horizontal distance XY = 550 metres and XZ = 270 metres. The surface levels of the boreholes are same. The depths to the coal seam at points X, Y and

Z are 125m, 175m and 179m respectively. Calculate the direction and rate of full dip of the seam.

Solution. The positions of boreholes X, Y and Z are roughly plotted.

Fall from X to Y = 175 − 125 = 50 metres.

Fall from X to Z = 179 − 125 = 54 metres.



Along XY the dip is at the rate of 50m in 550m = 1 in 11.

Along XZ the dip is at the rate of 54m in 270m = 1 in 5.

Let the point D along XZ is at the same level as at Y and at the rate of 1 in 5. Therefore 50m fall from X at point D will be at a distance of 50 x 5 = 250m.

YD is the strike line of the seam. A line XE at right angles of this strike line is the line of full dip.

Angle YXZ, $80^\circ - 52^\circ = 28^\circ$

In triangle XYD, $\tan\frac{D-Y}{2} = \frac{d-y}{d+y}\tan\frac{D+Y}{2} = \frac{550-250}{550+250}\tan\frac{180^\circ-28^\circ}{2}$

$= {}^3/_2 \tan 76^\circ = 1.50404 = \tan 56^\circ 23'$

Angle D = $56^\circ 23' + 76^\circ 00' = 132^\circ 23'$

Again, in triangle XDE, $\angle XDE = 180^\circ - 132^\circ 23' = 47^\circ 37'$

$\therefore$ Angle EXD = $90^\circ - 47^\circ 37' = 42^\circ 23'$

Hence *the direction of full dip* = S ($52^\circ - 42^\circ 23'$) W = S $9^\circ 37'$ W

Amount of full dip :

XE = XD cos $42^\circ 23'$ = 250 x 0.73865 = 184.6628 metres.

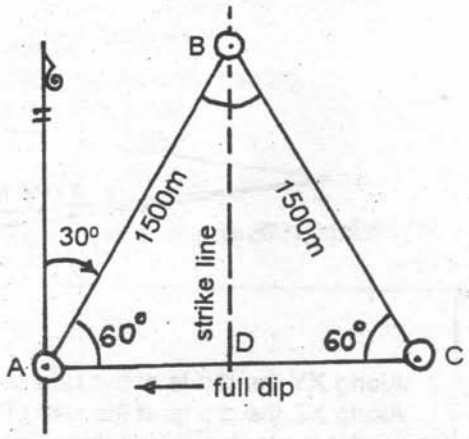
or, Dip along XE is 50m in 184.6628m = 1 in 3.69

$\therefore$ *Amount of full dip* = **1 in 3.69**

Example 32. A small colliery taking has been proved by three boreholes. The surface level at each bore and the depth to the seam proposed to be worked are respectively as follows :

	Surface level	*Depth to seam*
Bore A	70m A.O.D.	120m
Bore B	46m A.O.D.	66m
Bore C	94m A.O.D.	84m

Bore B is 1500m from A on a line bearing N 30°E and bore C, 1500m from B on line bearing S 30°E. Culculate the azimuth of the line of full dip and the rate of dip.



Solution : The positions of the three boreholes A, B and C are plotted to scale 1 cm = 300m, and these are joined to form the triangle ABC which is equilateral with sides 1500m long.

Now, the depth of the seam at each bore above or below ordnance datum is calculated -

Borehole	*Surface Level*		*Depth to Seam*	*Below or above O.D*
A	(70m A.O.D.)	-	(120m)	= – 50m
B	(46m A.O.D.)	-	(66m)	= – 20m
C	(94m A.O.D.)	-	(84m)	= + 10m

The Seam at A and B is 50m and 20 m below Ordnance datum respectively whereas at C, it is 10m above O. D.

Hence A is the lowest point and the fall from B to A = 50m – 20m = 30m.

Fall from C to A = 10m + 50m = 60m, and so D, the mid-point of CA has the same level as B (=20m). Thus BD is the strike line, running north and south, and CA, running due west, is the line of the full dip.

The rate of full dip = 60m in 1500m = **1 in 25.**

The direction of full dip = Due West = **an azimuth of 270°.**

Example 33. A very flatly dipping orebody has been probed by three vertical boreholes. The surface level of borehole collars and the depths at which the hanging wall of the orebody has been intercepted are as follows :

Borehole	*Surface level*	*Depth to seam*
A	10m abobe datum	87m
B	15m above datum	20m
C	25m above datum	25m

Line	*Bearing*	*Distance*
AB	N 30°E	300m
BC	S 45°E	500m

Calculate the azimuth of the line of full dip and the rate of dip in degrees. Assume that the orebody dips uniformly in this area.

Solution : By cosine Rule -

$AC^2 = AB^2 + BC^2 - 2AB \times BC \times \text{Cos}\,(30^0 + 45^0) = 300^2 + 500^2 - 2 \times 300 \times 500 \text{ Cos } 75^0 = 262354$

$AC = \sqrt{262354} = 512.20$ metres.

$$\text{Sin BCA} = \frac{300 \times \text{Sin } 75^0}{512.20} \qquad \therefore \text{ Angle BCA} = 34^0 27'$$

Angle BAC = $180^0 - (75^0 + 34^0 27') = 70^0 33'$

Let the R. L. of datum plane is 100 metres :

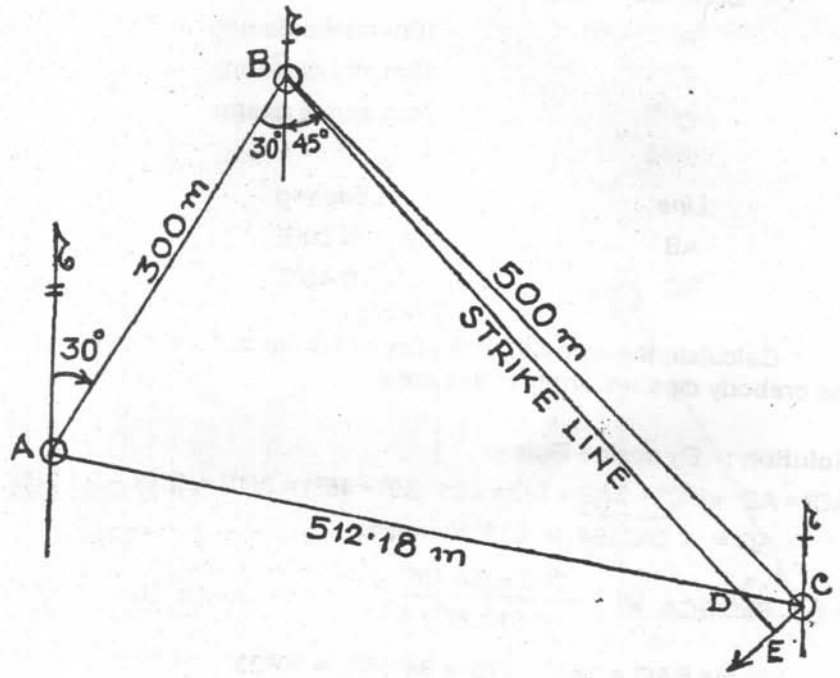
$\therefore$ The surface R. L. of borehole A = 100 + 10 = 110m
The surface R. L. of borehole B = 100 + 15 = 115m
The surface R. L. of borehole C = 100 + 25 = 125m

R. L of orebody at A = 110 – 87 = 23m
R. L of orebody at B = 115 – 20 = 95m
R. L of orebody at C = 125 – 25 = 100m

Fall from B to A = 95 – 23 = 72m in 300m
Fall from C to A = 100 – 23 = 77m in 512.20m

$\therefore$ A fall of 72m occurs in $\frac{512.20 \times 72}{77} = 478.94$m from A and at point D lying along AC. Point B and D having same level lying over the strike line of the orebody. BD is joined. CE

is perpendicular to BD produced. CE is the line of full dip.



CD = 512 – 478.94 = 33.26m

Again, $BD^2 = CD^2 + BC^2 - 2CD \times BC \times \cos 34^\circ 27'$

$= 33.26^2 + 500^2 - 2 \times 33.26 \times 500 \times 0.8246$

$= 223680$

$BD = \sqrt{223680} = 472.94$m.

$$\text{Sin CBD} = \frac{33.26 \times \text{Sin } 34^\circ 27'}{472.94} = 0.03978$$

$\therefore$ Angle CBD $= 2^\circ 17'$

Angle CBD $= 180^\circ - (34^\circ 27' + 2^\circ 17') = 143^\circ 16'$

Angle CDE $= 180^\circ - 143^\circ 16' = 36^\circ 44'$

Angle DCE $= 90^\circ - 36^\circ 44' = 53^\circ 16'$

Azimuth of the line of full dip CE $= 360^\circ - (45^\circ + 34^\circ 27' + 53^\circ 16') =$ **227° 17'**

Gradient of the line CE :

$$\text{CE} = \frac{33.26 \times \text{Sin } 36^\circ 44'}{\text{Sin } 90^\circ} = 19.89 \text{ metres.}$$

Difference of R. L from C to E = 100 – 95 = 5m

∴ Gradient = 5m fall in 19.89m = 1 in 3.978

Cot = 3.978 When $\theta = 14^0 07'$

Hence rate of dip in degrees $= 14^0 07'$

Example 34. The following information was obtained in the course of prospecting a certain seam of coal reserves -

Station	*Surface level* (metres A.O.D)	*Remarks*
X	60	Outcrop seam A
Y	240	Outcrop seam A
Z	90	Borehole depth to seam A = 300m

Y is 2700 metres from X on bearing N 33°W, and Z is 4500 metres from Y on a bearing N 50° E. Calculate the amount and direction of true dip of the seam.

Solution : Gradient of seam towards YX = (240 – 60)m in 2700 = 180m in 2700m = 1 in 15.

Gradient of seam towards YZ = 300 + (240 – 90) in 4500n = 450 in 4500m = 1 in 10.

$$\text{Tan}\ \frac{Z - X}{2} = \frac{15 - 10}{15 + 10}\ \text{Cot}\ \frac{ZYX}{2} = \frac{5}{25}\ \text{Cot}\ \frac{180^0 - (50^0 + 30^0)}{2}$$

$^1/_5 \cot 50^0 = 0.1678 = \tan 9^0 32'$

Now, $Z - X = 19^0 04'$ and $Z + X = 80^0 00'$ ∴ $Z = 49^0 32'$

Angle $X = 80^0 00' - 49^0 32' = 30^0 28'$

Amount of ture dip = 10 Sin $49^0 32'$ = **1 in 7.61**

Azimuth Direction of true dip = $50^0 + (90^0 - 49^0 32')$ = **$90^0 18'$ or S $89^0 32'$ E.**

Example 35. The surface levels, in metres, at the mouth of the shaft A and B, and the depths of the shafts from the surface are respectively as follows :

Shaft	*Surface level*	*Depth*
A	247.19 A. O. D.	200m
B	160.65 A. O. D.	150m

The co-ordinates of of the centres of the two shafts, in metres, are respectively as follows :

A	N 658.63	W 233.83
B	N 107.74	E 246.46

Find by calculation the length of cross-measure drift which connects the bottom of shaft A with the bottom of shaft B. What is the gradient of this drift ?

Solution : From the given data, the levels at bottom of the shafts are calculated -

Shaft	*Surface Level*	*Depth*	*Level at bottom*
A	247.19 A. O. D.	200m	47.19m A. O. D.
B	160.65 A. O. D.	150m	10.65m A. O. D.

∴ Difference of level at bottom of shafts A and B

= 47.19 – 10.65 = 36.54m fall from A to B

Length of AB = $\sqrt{(\text{Algebraic difference of Lat})^2 + (\text{Algb. diff. of dep})^2}$

= $\sqrt{(658.63 - 107.74)^2 + (233.83 + 246.46)^2}$

= $\sqrt{550.89^2 + 480.29^2}$ = 730.86m.

∴ *Plan length of drift* = **780.86 metres.**

Gradient of drift = 36.54 in 730.86 = **1 in 20.**

FAULT PROBLEMS

A *fault* may be defined as a fracture of the strata which has dislocated the original continuity of the beds, and has resulted in the beds being displaced either vertically or horizontalloy on one side of the fault.

Faults vary greatly in size, both longitudinally and vertically. They may extend across country for many kilometres and the vertical displacement of the fractured beds may range form centimetres to hundreds of metres. The vertical displaements is usually greatest near the centre of the fault, gradually reducing in both directions towards the ends until the displacement is nil.

On mine plans, the direction of a fault, as met with in the seam or seams, is indicated by a continuous red line. The assumed or conjectured positions of faults are shown by a broken lines. The direction of downward displacement is indicated by an arrow and the amount of displacement, where known, is stated in figures.

TYPES OF FAULTS

Faults may be divided into three main classes - (a) Normal faults, (b) Reversed faults, and (c) Transcurrent faults.

Normal Faults : These occur in regions subject to tensional and torsional stresses

and radial upthrusting of the earth's crust causing an extension of the surface of the region. In normal faults the strata are displaced in a downward direction, the fault plane hades or slopes forward in the direction of the displaced bed leaving an area of *barren ground.* The hade of a normal fault is commonly within 10^0 to 30^0

Normal Faults include : Step fault, trough fault, Stepped troughed fault and ridge fault.

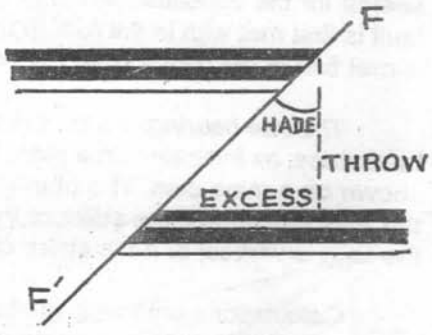
Reverse Faults : These are produced in regions subject to great lateral compression and in association chiefly with highly folded and deformed rocks. Unlike the normal faults, these hade to the upthrow side. The peculiarities found in this type of fault are -

(i) The ends of broken strata overlap and a vertical boring proves the same seam twice;

(ii) It hades at a grater angle from the vertical;

(iii) The older rocks overlie the younger rocks; and

(iv) It shortens the surface of the faulted region due to intense lateral compression.



Reverse Fault

Large reversed faults are known as **over-thrust** and the reversed ridge fault is **horst.**

Both normal and reversed faults may be called as dip, strike and oblique faults depending on their general direction.

When the general direction of the fault is parallel to the dip of the strata this is known as *transverse fault* or *dip fault.* In a *strike fault* or *longitudinal fault*, the general direction is parallel to the strike of the strata. A fault running at some angle between dip and strike of beds is known as *oblique faults.*

Transcurrent Faults : In faults where the plane is vertical or almost so and the movement of beds along the plane being entirely in a horizontal direction thus having no throw, are known as *transcurrent faults.*

(a) Fault plane : The fracture agianst which the rocks are relatively displaced is called the fault plane.

(b) Direction : The direction of the strike along the fault plane is known as the direction of a fault.

(c) Hade : The angle between the vertical and the fault plane is known as hade.

(d) Throw : The amount of vertical displacement of the bed is known as the throw.

(e) Want : This is the horizontal displacement of the beds leaving some barren ground

the extent of which depends on the head and the throw. Want is also known as shift or heave.

Want = Throw x tangent of hade

(f) Excess : In case of a reversed fault the fault-plane hades to the down upthrow side and the broken edge of the bed on the down-throw line underneath thus giving rise to an area of repetition of the bed. This is known as excess.

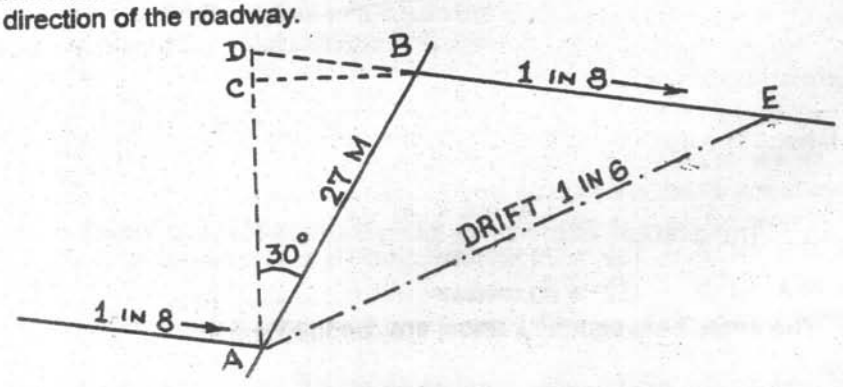
Both want and excess are lateral shifts or heaves.

The most distinguished feature of a normal fault is that the fault plane hades or slopes towards the displaced seam or stratum, and this gives an indication of the direction in which to search for the continuation of the seam or stratum on the other side of the fault. If a normal fault is first met with in the roof of the seam it is a *downthrow fault* and if, on the other hand, it is met first in the floor it is an *uphrow fault.*

The true bearing of a fault (known as the strike of a fault) is the direction of a level in the fault plane, as indicated on a plan. This should not be confused with the direction of a fault as shown on a mine plan. The plan shows the intersection of the fault plane and the seam and this intersection and the strike of the fault only coincide when the seam is level or the hade of the fault is vertical or if the strike of the fault is parallel to the strike of seam.

Calculations involving faults are very often associated with the determination of the length of a drift or tunnel to be driven through the fault to intersect the seam on the other side. All fault problems can be solved either trigonometrically or by the method of rate of approach. It is always preferable to plot the information given to scale so that a check on the calculated information is always available.

Example 1. A rodway dipping at a gradient of 1 in 8 in the direction of full dip of a seam strikes an upthrow fault bearing at right angles there to. The fault is subsequently found to have thrown up the seam a distance of 27 metres, measured from floor to floor on the hade of the fault. The hade of the fault is 30° from the vertical. Calculate the length of a cross-measures drift to win the seam, commencing at the lower side of the fault and rising at 1 in 6 in the same direction of the roadway.



Solution : The problem is illustrated in the figure. AB represents the meaurement of 27 metres along the head of the fault and the drift rising at 1 in 6 is indicated by AE.

Method 1. The drift rising at 1 in 6 makes an angle of $\tan^{-1} {}^1/_6 = 9^\circ 27'45'$ with the horizontal. The seam dipping at 1 in 8 makes an angle $\tan^{-1} {}^1/_8 = 7^\circ 07'30"$ with the horizontal.

The seam BE and the drift meeting at E make an angle BEA $= 9^\circ 27'45" + 7^\circ 07'30" = 16^\circ 35'15"$.

The angle BAE $= 90^\circ - (30^\circ + 9^\circ 27'45") = 50^\circ 32'15"$.

Therefore, the angle ABE $= 180^\circ - (16^\circ 35'15" + 50^\circ 32'15") = 112^\circ 52'30"$.

Now, in triangle BAE,

$$\frac{AE}{\text{Sin } 112^\circ 52'30"} \times \frac{AE}{\text{Sin } 16^\circ 35'15"} \quad \therefore AE \quad \frac{27 \times \text{Sin } 112^\circ 52'30"}{\text{Sin } 16^\circ 35'15"} = 87.14\text{m}$$

Therefore, **lengthe of drift = 87.14 metres**

Method 2. Vertical displacement of the seam by fault = AC = AB cos 30° $= 27 \times \cos 30^\circ = 23.382$ metres.

Vertical distance, AD = AC + CD $= 23.382 + \frac{27 \times \text{Sin } 30^\circ}{8}$

$= 23.382 + 1.688 = 25.07$ metres.

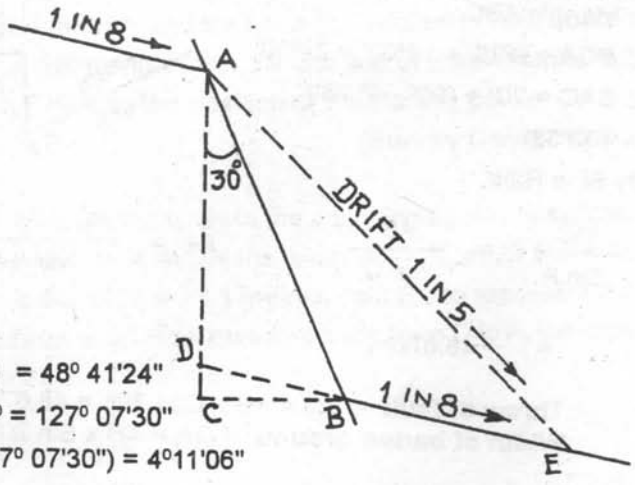
Effective grade of seam and drift $= {}^1/_8 + {}^1/_6 = {}^7/_{24}$

Plan length of drift AE $= 25.07 \times {}^{24}/_7 = 85.95$m.

$\therefore$ **Inclined length of drift** $= 85.95 \times \frac{\sqrt{37}}{6} =$ **87.14 metres.**

Example 2. A coal seam dips at 1 in 8. A roadway driven to the full dip meets a 15 metres down through normal fault hading at 30° to the vertical. The roadway is continued on the same bearing and on a slope of 1 in 5. What will be the length of the roadway driven to meet the seam on the other side of the fault ? **(Survey Comp. 1956)**

Solution : In the adjoining figure, AB is the fault plane AE is the drift dipping at 1 in 5. Vertical throw, AC = 15 metres.



Mehod 1. In triangle ACB, AB = AC x Sec 30° = 15 x Sec 30° = 17.32m. Slope of drift $= \tan^{-1} {}^1/_8 = 11^\circ 18'36"$. Slopeof seam $= \tan^{-1} {}^1/_5 = 7^\circ 07'30"$.

$\angle EAB = 90^\circ - (30^\circ + 11^\circ 18'36") = 48^\circ 41'24"$

$\angle ABE = (180^\circ + 7^\circ 07'30") - 60^\circ = 127^\circ 07'30"$

$\angle AEB = 180^\circ - (48^\circ 41'24" + 127^\circ 07'30") = 4^\circ 11'06"$

in triangle ABE, $AE = \dfrac{AB \times \sin 127°07'30''}{\sin 4°11'06''} = 189.24$ metres.

Hence, **the length of the roadway to meet the seam on other side of fault is 189.24 metres.**

Mthod 2. Width of the fault, $BC = 15 \tan 30° = 8.660$ metres.

$\therefore CD = \dfrac{8.660}{8} = 1.083$m.

Vertical distance, $AD = AC - CD = 15 - 1.083 = 13.917$m

Now, the rate of approach of AE and DE both dipping on the same direction $= {}^1/_5 + {}^1/_8 = {}^3/_{40}$

$\therefore$ Plan length of drift $AE = 13.917 \times {}^{40}/_3 = 185.56$m.

$\therefore$ **Inclined length of drift** $= 185.56 \times \dfrac{\sqrt{26}}{5} =$ **189.241 metres.**

Example 3. A roadway dirven on the full dip of a seam, which is 1 in 6, encounters a normal fault at B, bearing at right angles to the roadway. The hade of the fault is 20° from the vettical. A cross-measures drift BC, rising at 1 in 4, driven on the line of the roadway produced recovers the seam at C, 120 metres from B measured along the floor of the drift. The direction and rate of dip of the seam are the same on both sides of the fault. Calculate the throw of the fault and the width of the barren ground. **(Survey Comp. 1971)**

Solution : In figure, BC is the cross-measures drift rising at 1 in 4 and 120 metres long. BD is the vertical throw and DA is the horizontal displacement of width of the barren ground.

Gradient of seam = 1 in 6

$\therefore$ Inclination $= \tan^{-1} {}^1/_6 = 9°\,28'$

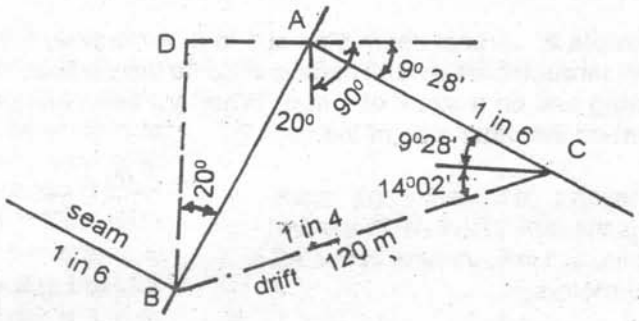
Gradient of drift = 1 in 4

$\therefore$ Inclinateion $= \tan^{-1} {}^1/_4 = 14°\,02'$

In triangle ABC,

$\angle BCA = 9°28' + 14°02' = 23°30'$

$\angle BAC = 20° + (90° - 9°\,28') = 100°32'$



By Sine Rule,

$$\frac{BC}{\sin A} = \frac{AB}{\sin C} \qquad \therefore AB = \frac{BC \times \sin C}{\sin A} = \frac{120 \times \sin 23°30'}{\sin 100°32'}$$

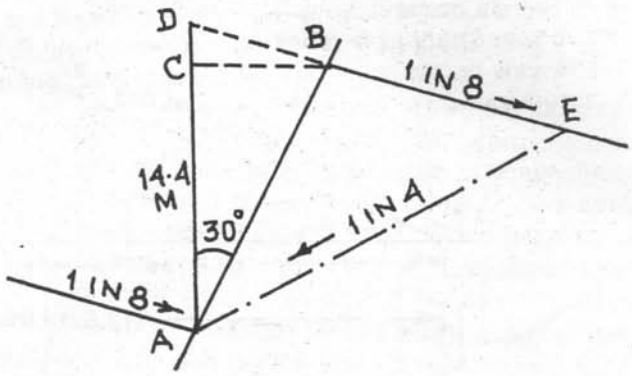
$= 48.670$m

Throw of fault $= BD = AB \times \cos 20° = 48.670 \times \cos 20° =$ **45.735m.**

Width of barren ground $= DA = AB \times \sin 20° = 48.670 \times \sin 20° =$ **16.646m.**

Example 4. In a seam dipping at 1 in 8 one upthrow fault is met with. The fault hades at 30° and has thrown the seam up by 14.4 metres. Calculate the length of a drift rising at 1 in 4 driven to meet the seam on the other side of the fault. Assume that the direction and rate of dip of the seam remains unchanged on the other side of the fault. **(Survey Comp. 1957)**

Solution : The given data are illustrated in figure. Considering the triangle ADE, we have DE and AE dipping at 1 in 8 and 1 in 4 in opposite directions and separated by a vertical distance equal to AD.



∴ Rate of approach = $^1/_4 + {}^1/_8 = {}^3/_8$

Vertical distance = AD

= AC + CD

AC = throw of fault = 14.4m

CAB = 30°

∴ Want of fault = BC = 14.4 tan 30° = 8.318m

$$\therefore \text{CD} = \frac{8.313}{8} = 1.0392 \text{ m.}$$

Now, the vertical distance between the seam and drift, AD = 14.4 + 1.0392 = 15.4392m.

Plan length of drift = 15.4392 x $^8/_3$ = 41.1712 metres.

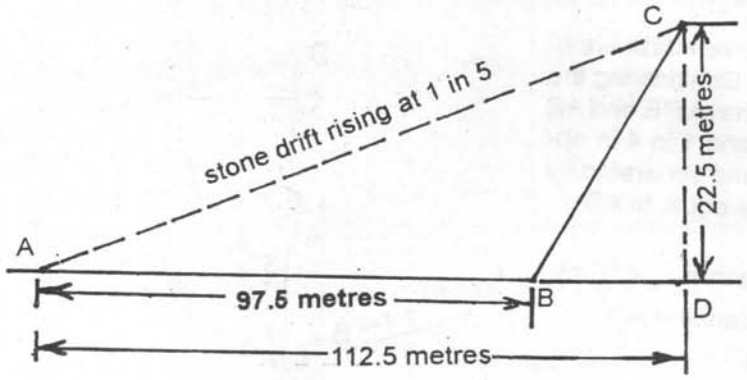
$$\therefore \textbf{Slope length of drift} = 41.1712 \times \frac{\sqrt{17}}{4} = \textbf{42.438 metres.}$$

Example 5. A level road strikes an upthrow fault running on the line of full dip. The vertical displacement of the fault is known to be 22.5 metres, and the hade $1^1/_2$ vertical to 1 horizontal. Calculate the length, in metres, of a stone drift rising at 1 in 5 in the line of road to strike the seam at the upper side of and at the fault. Find also the position of the starting points. Check by plotting to scale of a 1 cm = 10m. **(Survey Comp. 1982)**

Solution: The section is shown plotted to a reduced scale in the adjoining figure. First draw AB to represent the level road striking the fault at B. Make the width BD = $^2/_3$ of 22.5m = 15 metres and the throw DC (perpendicular to BD at D) = 22.5 metres. Join BC to represent the fault-plane and insert the seam on the upthrow side. Now measure back from D to A, the plan length of the stone drift, making DA = 5 x 22.5 = 112.5 metres.

The length of drift, AC = $\sqrt{112.5^2 + 22.5^2}$ = **11.473 metres.**

The starting point A is AD – BD = 112.5 – 15 = **97.5 metres** *back from the fault.*



Example 6. A heading AB driven direct to the rise in a seam at a graeient of 1 in 6 and in a direction due North strikes, at B, an upthrow fault bearing at right-angles to the heading and hading 30° from the vertical. From B, a cross-measures drift, driven in the same direction as the heading, has recovered the seam at a position 420 metres from the upper side of the fault as measured in the seam. The difference in level between the beginning and end of the drift is 265 mettres. Assume that the direction and rate of dip of the seam are the same on both sides of the fault and calculate : (a) the vertical displacement of the fault, and (b) the length and gradient of the drift . **(Survey Comp. 1983)**

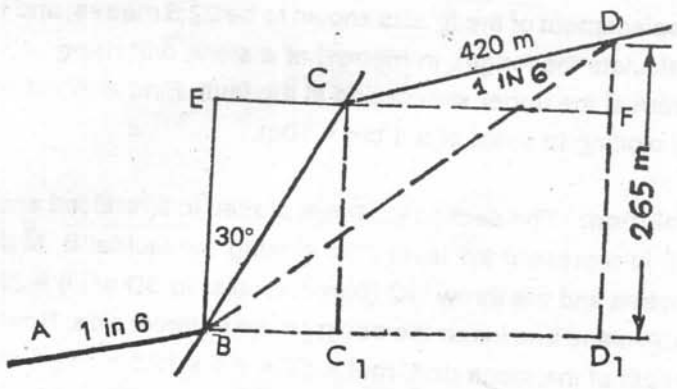
Solution : In the adjoining figure, CD = 420 metres.

$$\text{Plan length of CD} = \text{CF} = 420 \times \frac{6}{\sqrt{37}} = 414.28545 \text{ metres.}$$

$$\text{DF} = \frac{\text{CF}}{6} = \frac{414.28545}{6} = 69.047575 \text{ metres.}$$

(a) *Vertical displacement of the fault,* EB = DD_1 – DF = 265 – 69.047575 = **105.95243** metres.

(b) *Plan length of the drift* $BD = BC_1 + C_1D_1 = EC + CF = EB \tan 30^\circ + CF$ = 113.13319 + 414.28545 = 527.41864 metres.

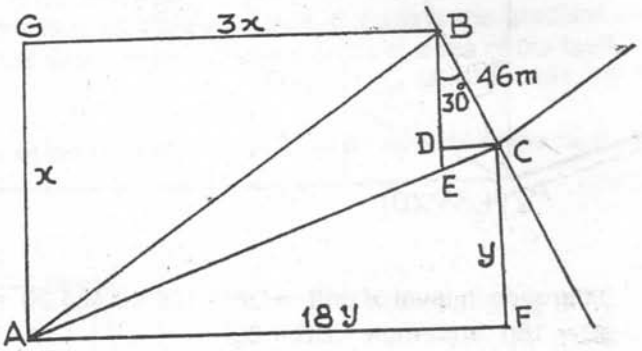


Gradient = 1 in $\frac{527.41864}{265}$ = **1 in 2.**

Inclined length of drift = Plan length $\times \frac{\sqrt{5}}{2} = 527.42 \times \frac{\sqrt{5}}{2}$ = **589.67 metres.**

Example 7. A heading rising due east at a gradient of 1 in 3 in a seam meets a fault bearing at right angles thereto with displacement down to east of 46 metres measured from floor to floor on the fault plane which is 30° from the vertical. The seam on the east side of the fault is to be won by driving from a point in the heading and in the same direction as the heading, a cross-measure drift rising at 1 in 18 so as to intersect the seam at its juncition with the fault. Calculate (a) the distance back from the fault at which the drift must be set way, and (b) the length of the drift. **(Survey Comp. 1984)**

Answer : In figure, let AB represents the heading rising at 1 in 3, and AC is the proposed crossmeasure drift rising at 1 in 18. The hade of the fault BC is 30° and the length of BC is 46 metres, therefore the vertical throw BD and DC :



BD = 46 x cos 30° = 39.83 metres.
DC = 39.83 x tan 30° = 23 metres.
EC is rising at 1 in 18 and the distance DC is 23m, hence the height ED = 23 ÷ 18 = 1.28.

The height BE = BD + DE = 39.83 + 1.28 = 41.11 metres.

The road AB and AE are converging at $^1/_3 + {}^1/_{18} = {}^{15}/_{54}$

The horizontal distance BG required to make up a difference of height of 41.11m is therefore 41.11m x $^{54}/_{15}$ = 148 metres.

(a) Since AB is rising at 1 in 3, the length of AB = $148 \times \frac{\sqrt{10}}{3}$ = 156 metres.

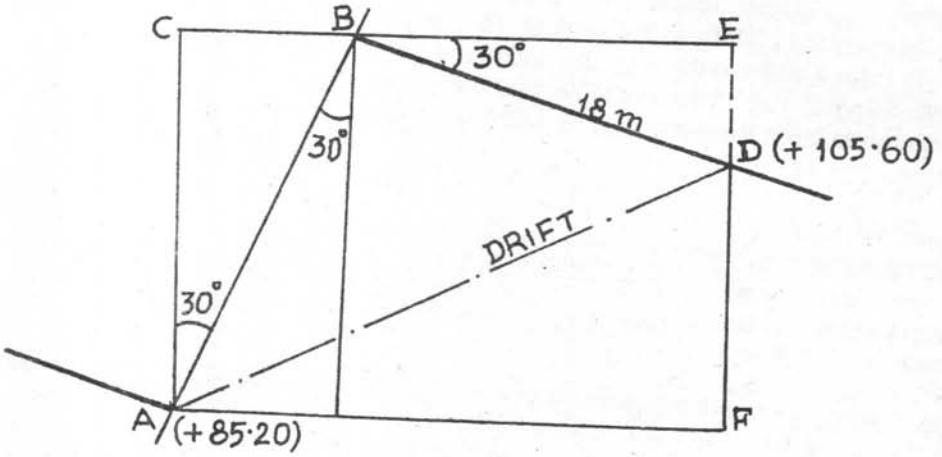
Therefore, *the drift must be started at A which is 156m back from the fault.*

(b) The horizontal length of the drift = AF = GB + DC = 148 + 23 = 171 metres.

The slope length of the drift = AC = $171 \times \frac{\sqrt{325}}{18}$ = **171.26 metres.**

Example 8. A roadway advancing on the line of full dip in a seam dipping due South at 30 degrees, intersected by an upthrow fault bearing East and West with a hade of 30 degrees from the vertical. From the point where the fault is struck a stone drift, bearing in the same direction as the roadway, is driven to prove the fault and reaches the seam at a point 18m - measured along the seam from the plane of the fault. The levels at the ends of the stone drift are 85.20m and 105.60m respectively above Ordnance Datum. Calculate the throw of the fault, and the length and inclination of the stone drift, assuming that the direction and rate of dip of the seam have been unaltered by the fault.

Solution : The figure below shows the details diagrammatically. AD is the drift.



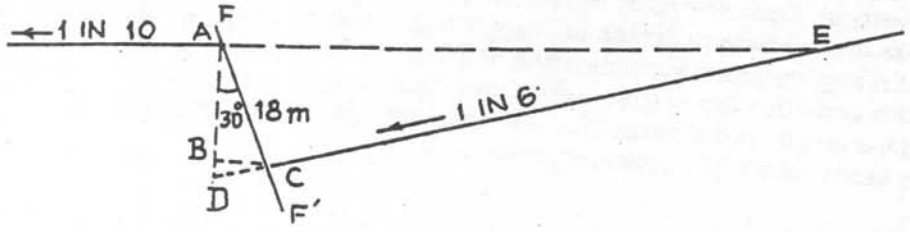
Difference in level of drift = DF = 105.60 - 85.20 = 20.40m.
BD = 18m and Angle EBD = 30° ∴ ED = 18 x Sin 30° = 9m.
Throw of fault, AC = DF + DE = 20.40 + 9 = **29.40m**
BE = 18 x Cos 30° = 15.59m
Want CB = throw x tan 30° = 29.40 x tan 30° = 16.97m.

$$\textit{Inclination of drift} = \frac{20.40}{15.59 + 16.97} = \frac{20.40}{32.56} = \frac{1}{1.6} = 1 \text{ in } 1.6$$

$$\textit{Length of drift} = \frac{32.56 \times \sqrt{(1.6)^2 + 1^2}}{1.6} = \textbf{38.40 metres.}$$

Example 9. The rate of full dip of a seam is 1 in 10. A haulage road advancing direct to the rise strikes a fault bearing at right angles to it. A drift is driven down the hade of the fault and proves the seam to be thrown down 18m measured from floor to floor on the angle of the hade, which is 30 degrees from the vertical. The direction of full dip of the seam beyond the fault is unaltered, but the rate of dip has increased to 1 in 6. Calculate the length of a cross-measure drift in the same direction and on the same gradient as the haulage roadway to win the seam beyond the fault.

Solution : The figure shows the details diagrammatically.



Want caused by the fault = BC = 18 x Sin 30° = 9m.

Vertical distance, AD = AB + BD = 18 Cos 30° + $^{9}/_{6}$ = 18 x 0.866 + 1.5 = 17.088m.

Effective grade of two lines diverging = $^{1}/_{6}$ - $^{1}/_{10}$ = $^{1}/_{15}$

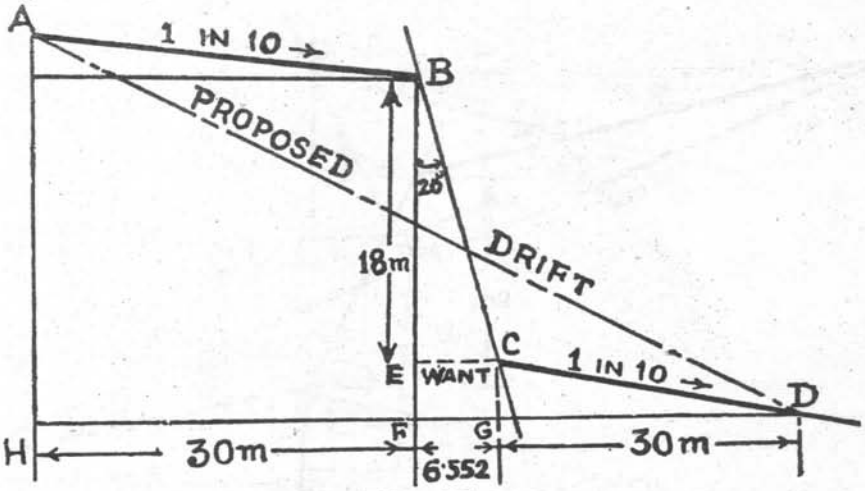
Plan measure of drift = 17.088 x $^{15}/_{1}$ = 256.320 m.

Length of drift at 1 in 10 = AE = $256.320 \times \frac{\sqrt{101}}{10}$ = **257.60 metres.**

Example 10. A roadway driven in the direction of the true dip of the seam (1 in 10) meets a fault bearing at right angle to the road. The seam is thrown down to 18m vertically. The hade of fault is 20°. A cross measure drift is to be driven from a point 30m to the rise of the fault, to meet the seam at a point 30m from the fault on the other side. Calculate the gradient and length of the drift assuming the dip of the seam to be the same on both sides of the fault.

(L. Min. E. 1981, 1985)

Solution : Let AB is the upthrow side of the fault and CD is downthrow side of the fault. AD is inclined length of the proposed drift.



Given, plan length of AB = 30m.
plan length of CD = 30m.
throw of fault, BE = 18m.

∴ Want of fault, EC = 18 tan 20° = 18 x 0.361 = 6.552m.
FD = EC + GD = 6.552 + 30 = 36.552m.

∴ Plan length of drift = HF + FG + GD = 30 + 6.552 + 30 = 66.552m.

Difference of level, from A to B = $^{30}/_{10}$ = 3m fall
from B to C = 18m fall
from C to D = $^{30}/_{10}$ = 3m fall.

∴ Difference of level from A to D = 3 + 18 + 3 = 24m fall.

Want caused by the fault = BC = 18 x Sin 30° = 9m.

Vertical distance, AD = AB + BD = 18 Cos 30° + $^{9}/_{6}$ = 18 x 0.866 + 1.5 = 17.088m.

Effective grade of two lines diverging = $^{1}/_{6}$ - $^{1}/_{10}$ = $^{1}/_{15}$

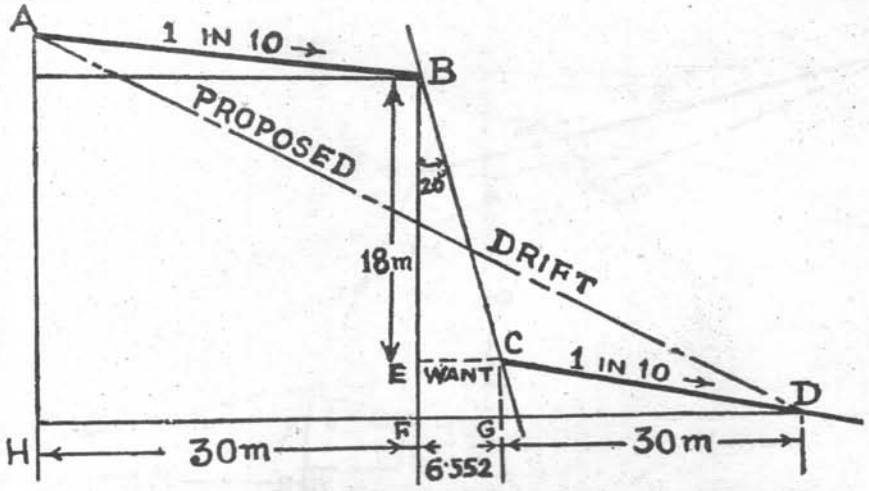
Plan measure of drift = 17.088 x $^{15}/_{1}$ = 256.320 m.

Length of drift at 1 in 10 = AE = $256.320 \times \frac{\sqrt{101}}{10}$ = **257.60 metres.**

Example 10. A roadway driven in the direction of the true dip of the seam (1 in 10) meets a fault bearing at right angle to the road. The seam is thrown down to 18m vertically. The hade of fault is 20°. A cross measure drift is to be driven from a point 30m to the rise of the fault, to meet the seam at a point 30m from the fault on the other side. Calculate the gradient and length of the drift assuming the dip of the seam to be the same on both sides of the fault.

(L. Min. E. 1981, 1985)

Solution : Let AB is the upthrow side of the fault and CD is downthrow side of the fault. AD is inclined length of the proposed drift.



Given, plan length of AB = 30m.
plan length of CD = 30m.
throw of fault, BE = 18m.

∴ Want of fault, EC = 18 tan 20° = 18 x 0.361 = 6.552m.
FD = EC + GD = 6.552 + 30 = 36.552m.

∴ Plan length of drift = HF + FG + GD = 30 + 6.552 + 30 = 66.552m.

Difference of level, from A to B = $^{30}/_{10}$ = 3m fall
from B to C = 18m fall
from C to D = $^{30}/_{10}$ = 3m fall.

∴ Difference of level from A to D = 3 + 18 + 3 = 24m fall.

$\therefore$ *Gradient of drift* = 24m in 66.552m or **1 in 2.773.**

Inclined length of drift $= \sqrt{AH^2 + HD^2}$

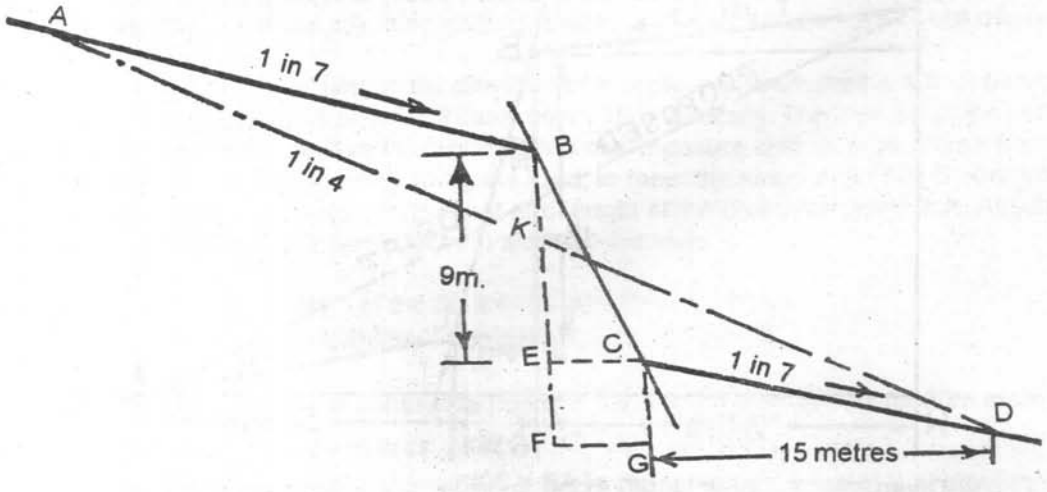
$= \sqrt{24^2 + 66.552^2}$ = **70.747 metres.**

Example 11. A roadway in a seam dipping at 1 in 7 on the line of the roadway meets a downthrow fault of 9 metres with a hade of two vertical to one horizontal. Calculate the length of a drift, dipping 1 in 4 in the line of the roadway, to win the seam at a point of 15 metres (plan measurement) from the dip side of the fault, and the distance (plan measurement) from the rise side of the fault, from which the drift must be set away. Assume the gradient of the seam to be uniform and the line of the fault at right angles to roadway.

Solution : In the figure AB represents the seam dipping at 1 in 7 and CD the seam on the downthrow side of the fault BC. AD represents the drift dippint at 1 in 4 and meeting the seam on the downthrow side at D.

Plan length of CD = 15m, and throw of fault, BE = 9m.

$\therefore$ Want caused by the fault, $EC = \frac{BF}{2} = \frac{9}{2} = 4.5$m.



Now, FD = EC + GD = 4.5 + 15 = 19.5m.

KD dips at 1 in 4, and so, $KD = FD \times \frac{\sqrt{17}}{4} = 19.5 \times \frac{\sqrt{17}}{4} = 20.10$m

$KF = \frac{FD}{4} = \frac{19.5}{4} = 4.875$m; again $EF = CG = \frac{GD}{7} = \frac{15}{7} = 2.142$m.

$\therefore$ KE = KF – EF = 4.875 – 2.142 = 2.733m.

BK = BE – KE = 9 – 2.733 = 6.267m.

BA and KA are both rising at 1 in 7 and 1 in 4 respectively starting off 6.267m vertically apart.

$\therefore$ Rate of approach $= {}^{1}/_{4} - {}^{1}/_{7} = {}^{3}/_{28}$

BK and KA will meet at a horizontal distance of $6.267 \times {}^{28}/_{3} = 58.492$m.

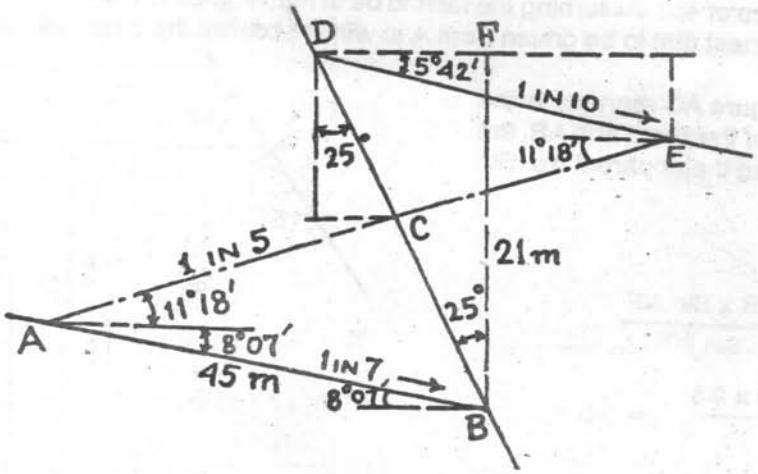
$\therefore$ *Starting point of the drift = 58.492 metres (plan length) from B on the rise side.*

Length of AK $= 58.492 \times \dfrac{\sqrt{17}}{4} = 60.29$m.

$\therefore$ *Length of drift,* AD = AK + KD = 60.29 + 20.10 = **80.39 metres.**

Example 12. A seam dipping due East at 1 in 7 strikes a reverse fault running North-South, with a hade of 25° and an upthrow of 21 metres. It is decided to drive a cross-measure drift, rising at 1 in 5 to intersect the seam on the eastern side, starting 45 metres back from the fault. Assuming that the seam on the eastern side is dipping due East at 1 in 10, calculate the length of the drift.

Solution : Cot of 1 in 5 = 11° 18'; and Cot of 1 in 7 = 8° 07'.



$\angle ABC = 90^\circ - (8^\circ\,07' + 25^\circ) = 56^\circ 53'$

$\angle ACB = 180^\circ - (11^\circ 18' + 8^\circ 07' + 56^\circ 53') = 103^\circ 42'$

$\angle CAB = 11^\circ 18' + 8^\circ 07' = 19^\circ 25'$

In triangle ABC,

$$AC = \frac{45 \times \sin 56^\circ 53'}{\sin 103^\circ 42'} = 38.79\text{m}$$

$$BC = \frac{45 \times \sin 19^\circ 25'}{\sin 103^\circ 42'} = 15.40\text{m}$$

In triangle BDF,

$BD = BF \times \text{Sec } 25^0 = 21 \times 1.10338 = 23.17\text{m}.$

$CD = BD - BC = 23.17 - 15.40 = 7.77\text{m}$

Again, $\angle EDF = \text{Cot of 1 in 10} = 5^0 42'$

Therefore, in triangle CDE,

$\angle CDE = 90^0 - (25^0 + 5^0 42') = 59^0 18'$

$\angle DEC = 11^0 18' + 5^0 42' = 17^0$

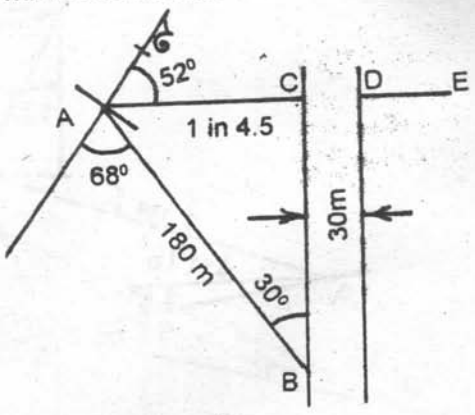
$$\therefore CE = \frac{CD \times \text{Sin } 59^0 18'}{\text{Sin } 17^0} = \frac{7.77 \times \text{Sin } 59^0 18'}{\text{Sin } 17^0} = 22.85\text{m}$$

The length of drift AE $= AC + CE = 38.79 + 22.85$

= 61.64 metres.

Example 13. The true dip of a seam is 1 in 4.5 due N 52° E. A roadway AB driven due S 68° E with a plan length of 180 metres has met a down throw fault at B with a vertical throw of 30 metres and hade of 45°. Assuming the fault to be at right angles to the true dip, calculate the length of the shortest drift to be driven from A to win the coal on the other side of the fault.

Solution : In the figure AC represents the direction of true dip of the seam and AB, the roadway, driven along the apparent dip. BC is the fault plane.

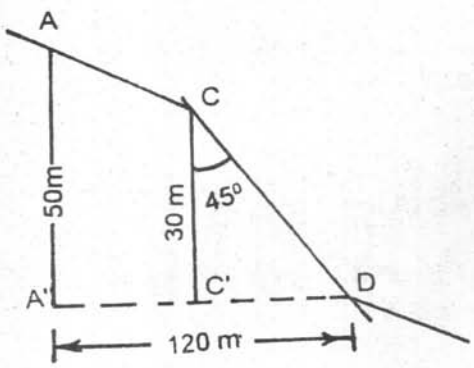


Plan

In triangle ABC,

$$AC = \frac{AB \times \text{Sin } 30^0}{\text{Sin } 90^0}$$

$$= \frac{180 \times 0.5}{1} = 90\text{m}$$



Section

Want of fault = C' D

$$= \frac{30}{\text{Tan } 45^0} = 30 \text{ metres.}$$

= plan length of CD.

The plan length of the shortest drift from A to win the coal on the otherside of the fault $= A'D = AC + CD = 90 + 30 = 120$ metres.

Again Fall from A to C $= \frac{90}{4.5} = 20m$

Fall from C to D = Throw of the fault = 30m.

Net fall from A to D = 20 + 30 = 50m.

Inclined length of the proposed drift AD

$= \sqrt{(AA')^2 + (A'D)^2} = \sqrt{120^2 + 50^2}$ **= 130 metres.**

Exampole 14. A roadway advancing due south at a gradient of 1 in 4 along the full dip of a coal seam is intersected by a normal fault bearing at right angles thereto. The displacement is 30 metres down to south. The fault hades at 30° from vertical. At a point in the roadway 100 metres (inclined measurement) back from the fault on the North side thereof, a cross-measure drift is driven due south to intersect the seam at a point 40 metres (inclined measurement) from the fault on the south side. Calculate the gradient of the cross-measure drift.

(Survey Comp. 1981)

Solution : In fighure, AF represents the drift. BD is the throw of the fault, 30 metres vertically.

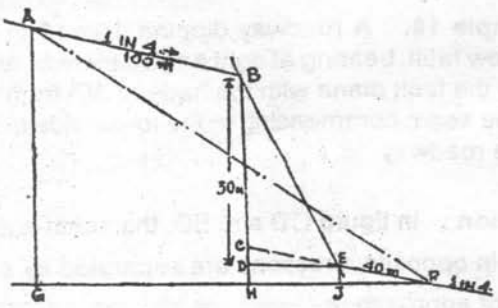
Want of fault = DE = 30 tan 30° = 17.32m

Plan length of AB = GH $= 100 \times \frac{4}{\sqrt{17}}$

= 97.015m.

Plan length of EF = JH $= 40 \times \frac{4}{\sqrt{17}}$

= 38.806m.



Hence plan length of drift = GH + DE + JF

= 97.015 + 17.32 + 38.806 = 153.141m.

Vertical fall from A to B $= \frac{97.015}{4}$

$= 24.254m$

Vertical fall from B to E = BD = 30.000m.

Vertical fall from E to F $= \frac{38.806}{4} = 9.701m.$

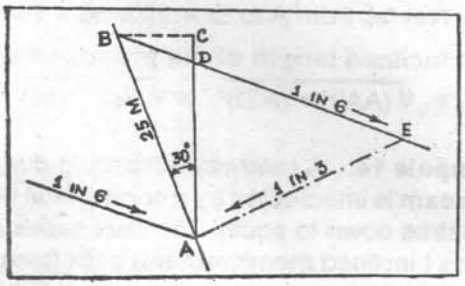
From A to F total fall = 63.955m.

Gradient of the drift AF = 63.955m in 153.141m = **1 in 2.394m**

Example 15. An underground roadway being driven along the full dip of the seam at 1 in 6 meets a reverse upthrow fault bearing at right angles thereto with a throw of 25m measured from floor to floor along the fault plane with angle of hade of 30° from vertical. Calculate the

length of the drift to win the seam commencing at the lower side of the fault rising at 1 in 3 in *the same direction as the roadway.*

Solution : In this case, the throw of the fault is the difference of level between A and B = AC as shown in figure. AB = 25 metres, and angle BAC = 30°.



Want of fault = BC = 25 x Sin 30° = 12.5m.
AC = 25 Cos 30° = 21.65m

$$CD = \frac{12.5}{6} = 2.083m$$

AD = 21.65 - 2.083 = 19.567m

Since the seam and drift dipping in opposite direction :

Rate of approach = $^1/_6 + ^1/_3 = ^1/_2$
Plan measure of drift = 19.567 x $^2/_1$
= 39.134m.

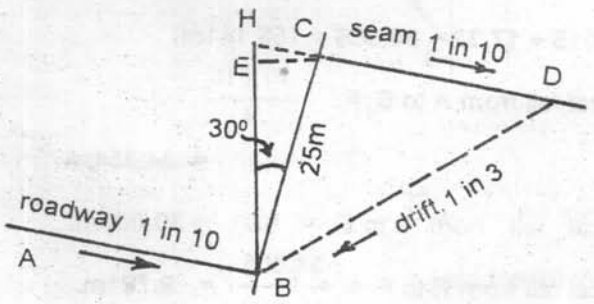
Actual length (inclined) of the drift at 1 in 3

$$= 39.134 \times \frac{\sqrt{10}}{3} = \textbf{41.251 metres.}$$

Example 16. A roadway dipping 1 in 10 in the direction of full dip of a seam strikes an upthrow fault, bearing at right angles thereto, with a throw of 25m measured from floor to floor along the fault plane with the hade of 30° from the vertical. Calculate the length of the drift to win the seam commencing at the lower side of the fault and rising 1 in 3 in the same direction as the roadway. **(Survey Comp. 1980)**

Solution : In figure CD and BD, the seam and the drift, dipping at 1 in 10 and 1 in 3 respectively in opposite directions are separated by a vertical distance BH.

Rate of approach = $^1/_{10} + ^1/_3 = ^{13}/_{30}$
Vervical distance = BH = BE + EH
In the 30° triangle CAB,
BC = 25, and EC = 12.5m



Throw of fault = BE = 125. x $\sqrt{3}$
= 21.65m

EH = 12.5 ÷ 10 = 1.25m
Hence BH = 21.65 + 1.25 = 22.90m.
Plan length of drift = 22.90 x $^{30}/_{13}$ = 52.847m

$$\textbf{Slope length of drift} = 52.847 \times \frac{\sqrt{10}}{3} = \textbf{55.706 metres.}$$

Example 17. A heading driven in coal seam is rising 1 in 4 and meets a down-throw fault bearing at right angles to the road. The distance along the fault plane to the seam on down-throw side is 46m. The hade of the fault is 20 degrees. At what point in the road a level road be started to meet the seam on the down-throw side at its point of intersection with fault. What is the length of this level road ? **(Survey Comp. 1985)**

Solution : The section is shown plotted in the adjoining figure. DA represents the heading at 1 in 4; AB is the fault hading at 20^0 and having a throw at 46m measured on the fault plane, DB is the level road to meet the seam on the down-throw side at B.

In triangle CAB.

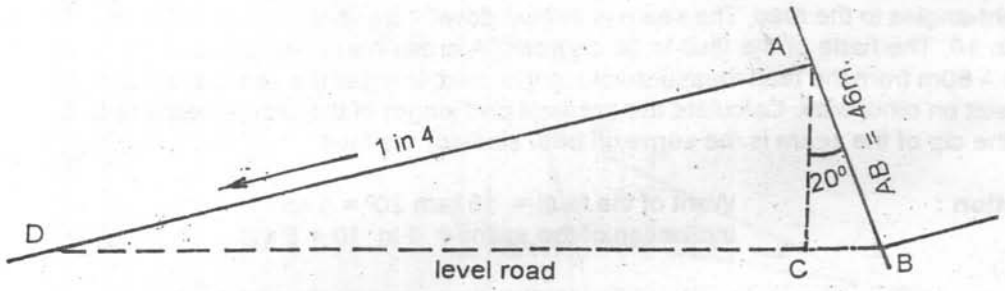
$$\frac{BC}{AC} = \text{Sin } 20^0 \qquad \therefore \; BC = 46 \times \text{Sin } 20^0 = 15.733\text{m.}$$

$$\frac{AC}{AB} = \text{Sin } 70^0 \qquad \therefore \; AC = 46 \times \text{Sin } 70^0 = 43.226 \text{ m.}$$

$\therefore$ DC = 43.226 x 4 = 172.904

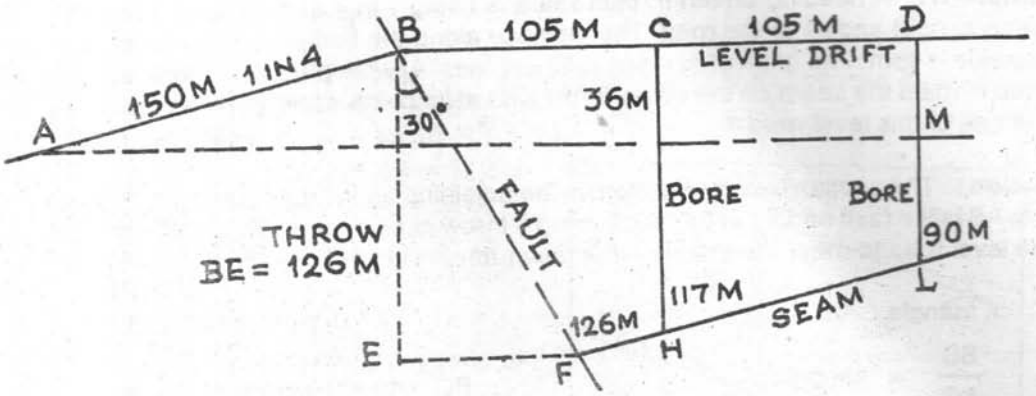
DB = DC + BC = 172.904 + 15.733 = 188.637 m.

$AD = \sqrt{AC^2 + DC^2} = \sqrt{43.226^2 + 172.904^2} = 178.225\text{m.}$



The starting point D is 178.225 metres back from fault and the length of the level road is 188.637 metres.

Example 18. A heading AB, 150 metres in length, bearing due East and rising 1 in 4, strikes at the point B a normal fault of unknown displacement bearing from North to South with a hade of 30^0 from the vertical. In a level exploring cross-measures drift driven due East from B, two boreholes C and D were put down from the floor and intersected the seam at depths equivalent to 81 metres and 54 metres respectively, below the level of A, the distances BC and BD being, respectively, 105 metres and 210 metres. Plot a section on the line AD showing the seam and the fault and from it measure the vertical displacement of the seam as a result of the fault. Plot the section to a scale of 1 cm = 25 metres. **(L. Mim. E. 1986)**



Solution : Draw AB rising at 1 in 4 and 150 metres long, and draw the fault plane BF at 30° from the vertical. From B draw BD to represent the level cross-measure drift and plot the boreholes at C and D, 81 metres and 54 metres deep, respectively, below the level of AM. Their total depth to H and L being 117 metres and 90 metres respectively. Produce LH backwards to cut the fault plane at F, and draw FE horizontal to represent the width of the fault. Then BE represents the vertical displacement of the seam at the fault.

By measurement of the adjoining plotted section : *Throw of faults = BE* = **126 metres.**

Example 19. A road being driven in the direction of true dip in a seam meets a fault bearing at right-angles to the road, The seam is thrown down 15m vertically. The true dip of the seam is 1 in 10. The hade of the fault is 20 degrees. A cross-measure drift is to be driven from a point A 60m from the fault measured along the road, to meet the seam at a point B 30m from the fault on other side. Calculate the gradient and length of the cross-measure drift. Assume that the dip of the seam is the same on both sides of the fault.

Solution : Want of the fault = 15 tam 20° = 5.4525m
Inclination of the seam = 1 in 10 = 5°42'

Plan length of the seam at upper side from the starting point of drift (or for 60m inclined length) = 60 cos 5°42' = 59.70 metres.

Plan length of the seam for 30m inclined length from the fault to point B at downthrow side = 30 cos 5°42' = 29.85 metres.

Plan length of the drift AB = 59.70 + 5.46 + 29.85 = 95.01m.

Again, the difference in level from the point A to fault = $\frac{59.70}{10}$ = 5.970m.

Difference in level from the fault to point B = $\frac{29.85}{10}$ = 2.985m.

Throw of the fault = 15m.

∴ Difference in level from A to B = 5.970 + 15 + 1.985 = 23.955m.

Tan (dip of drift) $= \frac{23.955}{95.01} = 0.2521313$

Therefore **dip of drift** = 14°09' or 1 in 3.97

Length of drift = 95.01 sec 14°09' = 97.98m

[or length of drift = $\sqrt{95.01^2 + 23.955^2}$ = 97.98m.]

Example 20. The rate of true dip of a coal seam is 1 in 3 in a direction due South. A road is being driven in a direction N 35° W and encounters an upthrow fault which has bearing of N 20°E, and a hade of 30 degrees. An exploratory road is driven up the face of the fault in a direction N 70°W and encounters the seam after driving 50 metres.

A cross-measure drift is to be driven from a point 200 metres back from the fault at a gradient of 1 in 2.5. Find the length of this cross-measure drift to the point of intersection with the seam. How far is this point of intersection from the fault plane ?

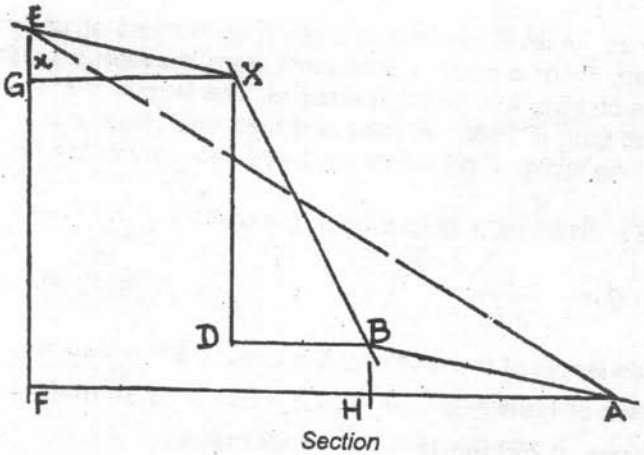
Solution : True dip of the seam = 1 in 3 in direction due South. Hence the apparent dip in direction N 35°W -

Tan apparent dip
= $^1/_3$ cos 145° =
= – 0.27305

or, apparent dip
= – 15°16'

Throw of fault
= 50 cos 30°
= 43.3m;

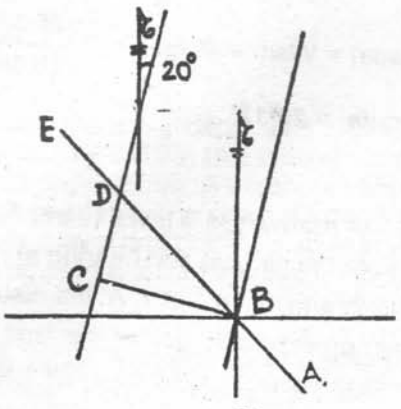
Want = 50 sin 30°
= 25m.



Section

In triangle BCD :
CB = 25m, ∠ CBD = 35° and
∠ BCD = 90°
Therefore, BD = 25 Sec 35° = 30.5m

In triangle AHB :
AH = 200 Cos 15°16' = 192.9m
BH = 200 Sin 15°16' = 52.66m
Also DB = 30.5m
and DX = 43.3m
AF = AH + DB + GX



Plan

$= 192.9 + 30.5 + x \text{ Cot } 15°16'$

$= 223.4 + x \text{ Cot } 15°16'$

FE = BH + DX + GE =

$52.7 + 43.3 + x - 96.0 + x$

But AF = 2.5 FE

$\therefore \quad 223.4 + x \cot 15°16'$

$= 2.5\ (96+x) = 240 + 2.5x$

or $\quad x\ (\cot 15°16' - 2.5) = 240 - 223.4$

or $\quad x\ (3.66 - 2.5) = 16.6 \qquad \therefore\ x = \dfrac{16.6}{1.16} = 14.3\text{m}$

$EX = x \text{ cosec } 15°16' = 14.3 \text{ cosec } 15°16' = 54.3\text{m}$

The cross-measure drift intersects the seam at a distance **54.3 metres from the fault plane** measured along the seam.

Example 21. A level roadway in a seam encounters an upthrow fault bearing at right-angles to the road, From a point A 50m back from the fault a cross-measure drift is driven at an inclination of 1 vertical to $3^1/_2$ horizontal. The length of this drift to the seam on the upthrow side of the fault is 250m. A road is driven back from this point to the fault, the distance of driving being 165m. Find the throw, hade and want of the fault.

Solution : To find the inclination of the drift -

$\tan \theta = \dfrac{1}{3.5} \qquad \therefore$ Inclination, $\theta = 15°57'$

Plan length of the drift $= 350 \times \text{Cos } 15°57' = 240.38\text{m}.$

Want of fault $= 240.38 - (165 + 50) =$ **25.38 metres**

Throw $= 250 \text{ Sin } 15°57' =$ **68.69 metres**

$\text{Tan (hade)} = \text{Want} \div \text{Throw} = \dfrac{25.38}{68.69}$

Hade = 20°17'

Example 22. A roadway in a level seam, advancing due North meets a normal fault with a hade of 30° from the vertical and bearing at right angles to the roadway. An exploring drift is set off due North and rising 1 in 1. At the distance of 12.3m, as measured on the slope of the drift, the seam on the north side of the fault is agian intersected. Find, by calculation, or by drawing to a suitable scale, the throw of the fault and the width of the barren ground. If the drift had been driven 1 in 4 (in place of 1 in 1), what would be the length of this drift ?

Solution : From the figure -

CX = 12.3 Sin 45° = 8.697m;

CX = DY = **Throw = 8.697m**

BY = **Width of barren ground**

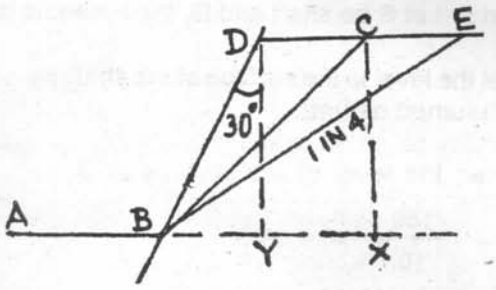
= 8.697 tan 30° = **5.02m**

If the drift was 1 in 4,

the inclination = 14° 02'

∴ **Length of drift** (1 in 4)

= 8.679 ÷ Sin 14° 02' = **35.86m**



Example 23. A roadway advancing due East in a seam dipping 1 in 6 due west meets at a point A an upthrow fault which bears N and S and hades at 30° from the vertical. The roadway is continued in the same direction but at a gradient of 1 in 4 from A until it intersects the seam again at B. The seam is found to dip at the same rate on both sides of the fault. The length of the drift AB is 120.6 metres. Calculate the horizontal width of the barren ground and the vertical displacement of the seam due to the fault.

Solution : Inclination 1 in 6 = 9°28'

Inclination 1 in 4 = 14°02'

In triangle AXB,

∠ AXB = 30° + 90° + 9°28' = 129°28'

∠ BAX = 90° – (30° + 14° 02') = 45°58'

∠ ABX = 14°02' – 9°28' = 4°34'

$$\frac{AX}{\text{Sin } 4°34'} = \frac{AB}{\text{Sin } 129°28'}$$

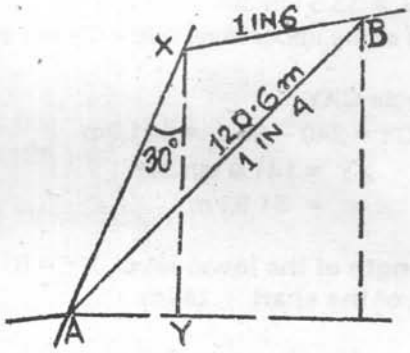
$$AX = \frac{120.6 \times \text{Sin } 4°34'}{\text{Sin } 129°28'} = 12.44 \text{ metres}$$

Width of barren ground = AY = 12.44 Sin 30°

= **6.22 metres.**

Vertical displacement = XY = 12.44 Cos 30°

= **10.77 metres.**

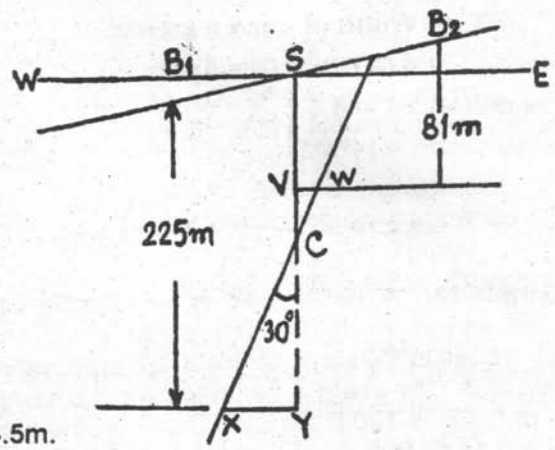


Example 24. A shaft is being sunk and cuts a fault plane at a depth 98.1 metres. The fault is a downthrow to the west and hades at 30° . A borehole 150m (measured horizontal) to the west of the shaft indicates a seam 1.8m thick at a depth 225m. A borehole 165m due east of the shaft indicates that the seam is 81m below the surface. The seam is known to be level. It is intended to continue sinking the shaft and drive a level drift due West to intersect the seam on the West side of the fault. What will be the required depth of the shaft ? What will be the length of the level drift ?

At what depth must an inset be made so that a level drift can be driven due East to intersect the seam on the East side of the fault ? What will be the length of this level drift ? The surface rises at 1 in 10 in a direction West to East.

Solution : Let S be shaft and B_1 the borehole to the West and B_2 the borehole to the East.

Let the level to the surface at the shaft be the assumed datum.



Then the level of the surface at B_1

$$= - \frac{150}{10} = -15\text{m}$$

The level of the surface at B_2

$$= + \frac{165}{10} = +16.5\text{m}.$$

Therefore the depth of the seam at B_1
$= -225 - 15 = -240$m.
Depth of the seam at B_2 $= 81 + 16.5 = -64.5$m.

In triangle CVW :
CV = 98.1 – 64.5 = 33.6m $\quad \angle VCW = 30^\circ$.
$\therefore$ VW = 33.6 tan 30° = 19.4m
Length of the upper level drift = **19.4m** and the *inset must be made at depth of* **64.5m.**

In triangle CXY :
CY = 240 – 98.1 = 141.9m $\quad \angle XCY = 30^\circ$
$\therefore$ XY = 141.9 tan 30°
= 81.92 m.

The length of the lower level, XY = 81.92m
Depth of the shaft = 240m

Example 25. A seam dipping at 1 in 4 in the direction S 30° W encounters a downthrow normal fault which runs in East-West direction. The vertical displacement of the fault is 50m and it hades at 30°. On the downthrow side of the fault the seam dips at 1 in 6 in the direction S 45° W. A stone drift dipping at 1 in 2 in the direction due south is to be driven from the upthrow side of the fault from a point 24m from the fault to touch the seam on the downthrow side. Calculate the length of the stone drift.

Solution : Let A be the starting point of the drift on upthrow side of the fault at 24m from the fault and let its R. L. be say 500m.

Since the seam is diping at 1 in 4 in a direction S 30° W upthrow side of the fault -
R. L. of B along the seam

$$= 500 - 24 \times \frac{1}{4} \cos 30^\circ$$

= 500 – 5.196 = 494.804m

R.L. at C on the downthrow side of the fault

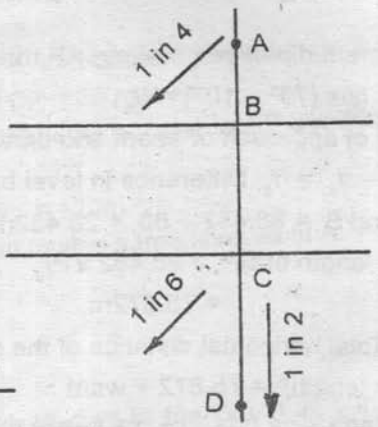
= 494.804 – 50 = 444.804m

Want of fault, BC = 50 tan 30° = 28.867m

If D is the point where the drift from A (upthrow side) meets the seam on downthrow side of the fault,

$$\text{R. L. of D} = \text{R. L. of C} - \frac{CD}{6} \cos 45^\circ$$

$$= 444.804 - 0.11785\ CD$$



Since the drift dips at 1 in 2.

$$\text{R. L. of D} = 500 - \frac{AD}{2} = 500 - {}^1/_2\ (AB + BC + CD)$$

$$= 500 - {}^1/_2\ (24 + 28.87 + CD)$$

$$= 500 - 26.435 - \frac{CD}{2} = 473.565 - \frac{CD}{2}$$

Hence 444.804 – 0.11785 CD = 473.565 – 0.5 CD

or, CD (0.5 – 0.11785) = 473.565 – 444.804 = 28.761

$$\text{or, } CD = \frac{28.761}{0.38215} = 75.27\text{m}$$

Horizontal length of drift = 24 + 28.87 + 75.27 = 128.14m

Inclined length of drift $= 128.14 \times \frac{\sqrt{5}}{2}$ **= 143.26 metres.**

Example 26. A level driven in a coal seam dipping at 1 in 4 in direction S 10° W touches a downthrow fault of 40m vertical displacement with a hade of 30°. Strike of fault is N 20° W. Draw a plan and section to show the drift driven from A (upthrow side) in a direction S 70° W at a gradiant of 1 in 2 to touch the seam on downthrow side. Calculate the length of the drift.

Solution : Let DA be the level roadway driven in the seam dipping 1 in 4 in direction S 10° W, and touches the fault at A. Hade of fault is 30°

Throw of the fault = 40m = AA'

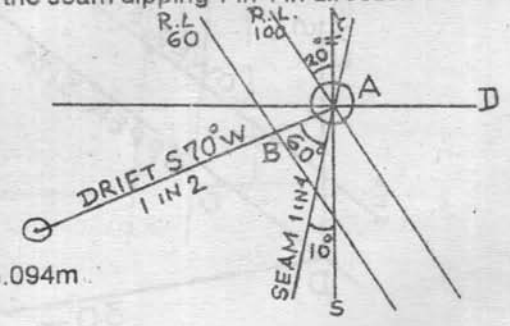
Want = A' B = 40 tan 30° = 23.094m.

Let R.L. of A = 100m.

∴ R. L. of B = 100 – 40 = 60m.

Since the drift dips at 1 in 2,

Fall in drift along AB' for a horizontal distance 23.094m



(Want) $= 23.094 \times \frac{1}{2} = 11.548$m

Hence R.L of drift at point B'

$= 100 - 11.548 = 88.452$m

Apparent dip of seam along AP (drift)

$= {}^1/_4 \cos (70^\circ - 10^\circ) = {}^1/_8$

Rate of approach of seam and drift

$= {}^1/_2 - {}^1/_8 = {}^3/_8$ Difference in level between B' and B $= 88.452 - 60 = 28.452$m.

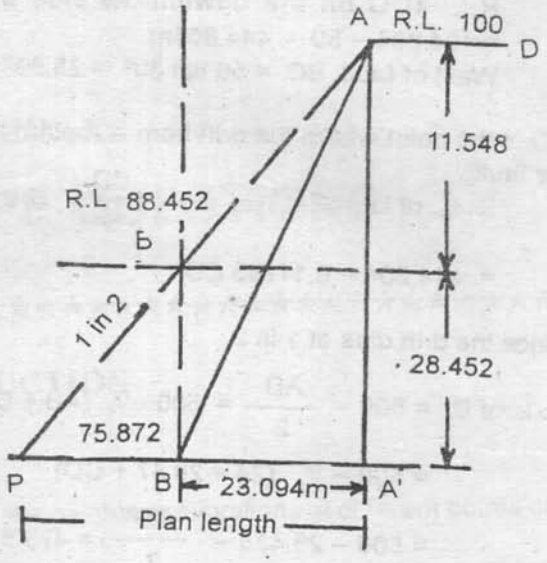
Plan length of BP $= 28.452 \times {}^8/_3$

$= 75.872$m.

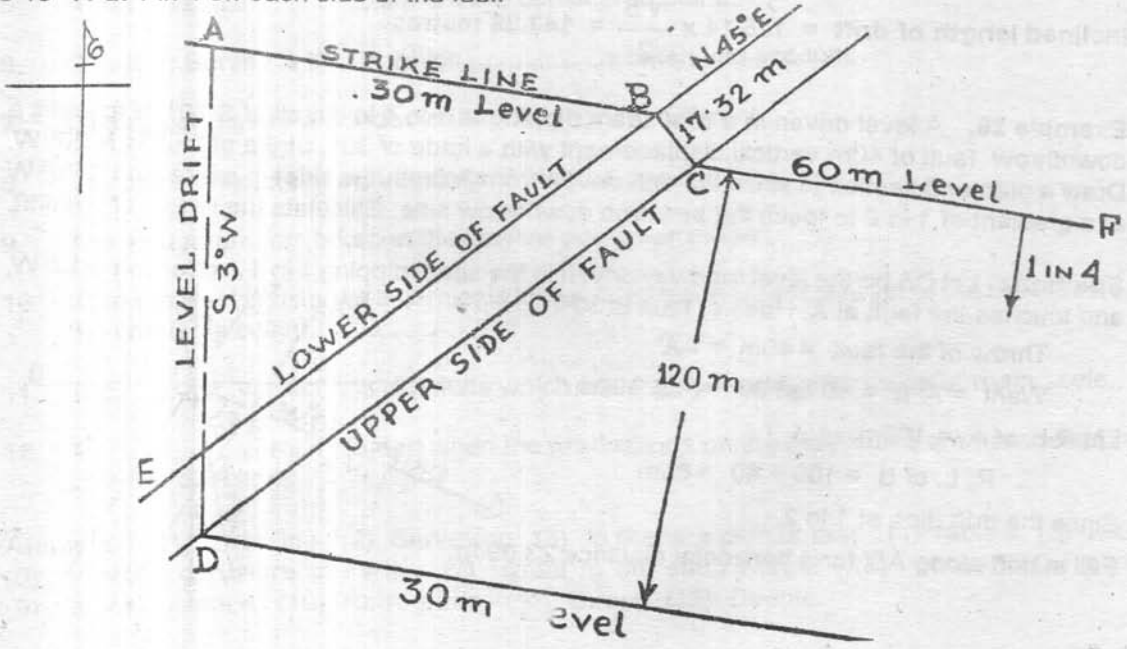
∴ Total horizontal distance of the drift (plan length) $= 75.872 +$ want

$= 75.872 + 23.094 = 98.966$m.

Inclined length of drift $= 98.966 \times \frac{\sqrt{5}}{2}$ **= 110.64 metres**



Example 27. A roadway advancing in the direction S 80° E on the strike of a coal seam meets a fault which bears N 45° E hade at 30° from the vertical and throw up the measures 30 metres vertically to the south-east. The distance AB is 120 metres and the strata dip towards S 10° W at 1 in 4 on each side of the fault



Draw to a scale of 1cm = 20m, a plan showing the roadway and fault, and from it ascertain the bearing and length of a level cross-measure drift to be driven from the point A to win the coal seam immediately on the upper side of the fault.

Solution : The plan is plotted to a scale 1cm = 20m in which AB is the level advancing in the direction of S 80° E meets the fault at B on the lower side. The line of fault BE runs along N45°E.

Vertical throw of the fault = 30m, and hade = 30° .

∴ Horizontal shift, BC = 30 x tan 30° = 17.32 metres.

BC is at right angles to BE and CD is parallel to BE to represent the seam on the upper side of the fault. Difference of levels between AB and CF = 30 metres.

As the seam dips at 1 in 4 towards S 10° W, DH is the level parallel to CF but 30m vertically below CF, thus DH having same level as at A.

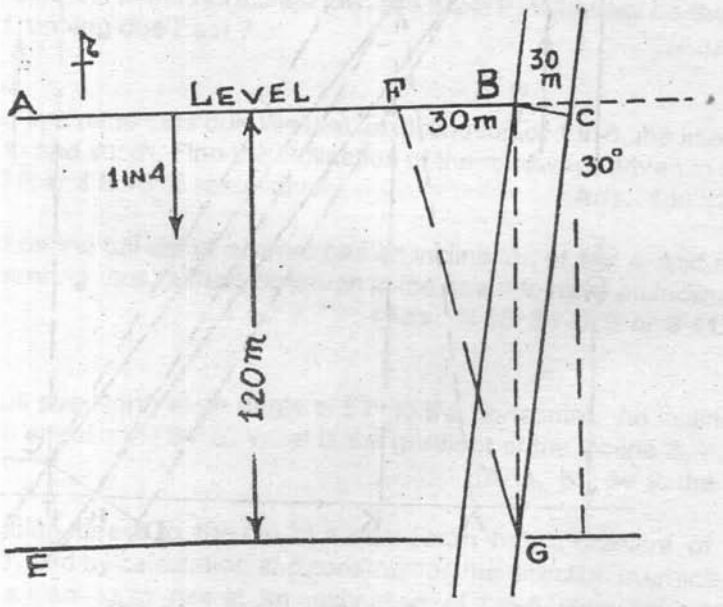
The horizontal distance between CF and DH = 30 x $^4/_1$ = 120m.

D is the point immediately on the upper side of the fault having same level as at A. AD is the drift to be driven.

Bearing of AD = S 3° W

Length of AD = 132 metres.

Example 28. A level is driven to the East in a seam which dips to the South at 1 in 4 and it meets an upthrow fault having a throw of 30 metres, a hade of 30° , and a strike of North and South. A level cross-measure drift is to be started at a point F, which is 30 metres back along the level from the fault, to reach the same seam on the upthrow side of the fault. Find the length and bearing of the drift.



Solution : The plan showing the level AB, the fault and the level cross-measure drift FG.

As the seam dips 1 in 4 and the throw of the fault is 30m, the horizontal distance from the line AB to the parallel line through E = 30 x 4 = 120m.

Horizontal shift = 30 x tan 30° = 17.320m.

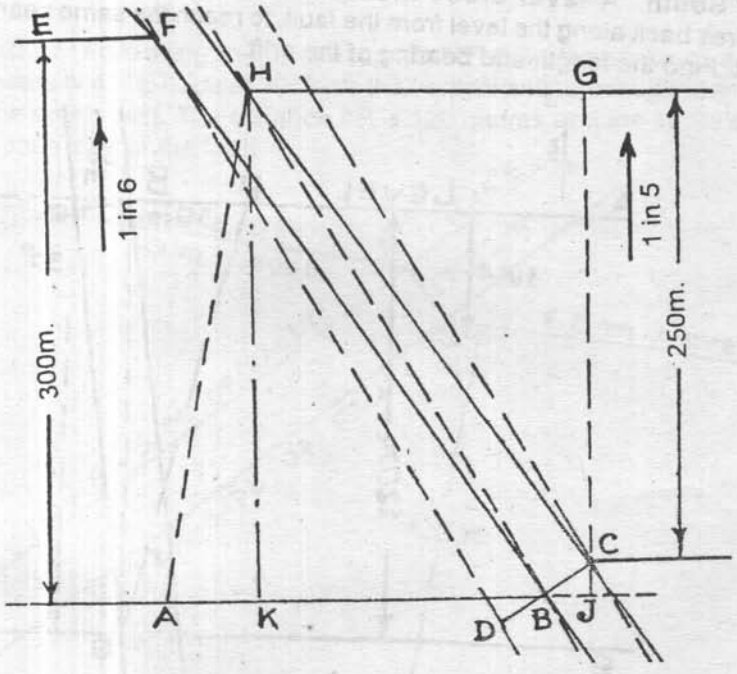
The bearing of the fault in the seam = S $\tan^{-1} \frac{17.320}{120}$ W = S 8° 13'W or N 8° 13' E

The bearing of level drift FG = S $\tan^{-1} \frac{30}{120}$ E = **S 14° 02'E**

Length of drift = $\sqrt{120^2 + 30^2}$ = **123.69 metres.**

Example 29. A level AB is driven due East in a seam which dips North at 1 in 6 and it meets an upthrow fault having a vertical displacement of 50 metres, the hade being 30° and the strike S 30° E. the dip of the beds on the upthrow side of the fault is 1 in 5 due North. Find the length of a level drift starting 200 metres back along the level from the fault and state the bearing of the drift.

Solution : The plan showing the level AB, the fault and the level drift AH. The throw of the fault being 50 metres and the hade 30°



∴ The horizontal distance, BC = 50 tan 30° = 28.870m.

The horizontal distance CG = 50 x 5 = 250m.

HG is parallel to AB, both are strike lines of the coal seam having same levels.

Now, BC = 58.87m and the angle CBJ = 30°

$\therefore$ CJ = 28.87 Sin 30° = 28.87 x 0.5 = **14.435m.**

and BJ = 28.87 Cos 30° = 28.87 x 0.866 = 25m.

$\therefore$ JG = 250 + 14.435m = 264.435m = HK

GH = 264.435 tan 30° + 25 = 152.68 + 25 = 177.68m.

AK = AJ - KJ.

= (AB + BJ) – GH = (200 + 25) – 177.68 = 47.32m.

Therefore, the length of drift from A to H on the same level = $\sqrt{AK^2 + HK^2}$

= $\sqrt{47.32^2 + 264.435^2}$ = 268.64m.

Bearing of the drift = N $\tan^{-1}\frac{47.320}{264.435}$ E = **N 10° 09' E.**

EXERCISES

1. What is understood by full dip and apparent dip ? The apparent dip in a direction S 30°W is 1 in 5, and that in a direction S 45°E is 1 in 3. Find the rate and direction of full dip. Check by plotting. **(Ans.** 1 in 2.829, S 22° 33'E)

2. A bed of mineral dips 47° to the horizontal towards S 24° E. What will be the gradient of a road in the bed running due East ? **(Ans.** 1 in 2.29)

3. The seam worked in a mine dips due West at an inclination of 1 in 6, the line of the level course being north and south. Find the inclination of the roadways driven in the seam at bearings of N 30° E and S 45° E respectively. **(Ans.** 1 in 12, 1 in 8.48)

4. A roadway driven on the full dip of a seam has an inclination of 1 in 4, and a bearing of S 25° E. At what bearing must a road be driven in the seam to have an inclination of 1 in 10 ? **(Ans.** N 88° 34'42"E or S 41° 25'18"W))

5. A mineral vein dips due North at an angle of 67° to the horizontal. An incline is set off along the vein in a direction N 24° E. What is the gradient of the incline ? **(Ans.** 65° 04' to the horizontal)

6. A roadway advancing direct to the dip in a coal seam has a gradient of 1 in 4 in an azimuth of 205° 30'. find by calculation and construction the direction in which a roadway should be set away so as to rise at an inclination of 1 in 8. Give your answer as a quadrant bearing. **(Ans.** S 85° 30'E or S 34° 30'W)

7. In two drifts, AB and AC, starting from the point A, AB is going N 24° E and AC N 37° W, the strata are seen to rise on going at 1 in 9 in AB and 1 in 5.5 in AC. What is the direction and amount of dip of the strata ? **(Ans.** S 28° 57'E, 1 in 5.445)

8. The full dip of a seam 1 in 4 in the direction N 12° 30'E, Find the bearing of a road in which the dip is 1 in 8. **(Ans.** N 72° 30'E and N 47° 30'W)

9. The azimuth of a heading AB in a coal seam is 110° 30' and its inclination is 5° 15'. If the strike line AC is 40° 35', calculate the rate of full dip and the quadrant bearing of full dip. **(Ans.** 1 in 10.23; S 49° 25'E or N 49° 25'W)

10. A stratum is seen to dip on two exposed faces of a quarry, the apparent dips being (a) 30° in the direction S 45° E. (b) 45° in the direction S 40° W. Calculate the direction and rate of full dip. **(Ans.** S 13° 48'W, 1 in 0.8973)

11. A drift is to be driven from a given point to pit C, on the other side of a river. The bearing of C from a point A is N 42° E, and after running a line AB by the river side for 200 metres in the direction N 85° E, its bearing again from B is N 30° W. What bearing must you set off a drift from a point D, 80 metres from A, along the line AB, so as to hit C, and what will be its length ? **(Ans.** N 19° 34'E, 142.90m)

12. A coal seam is met in a shaft 'A' at 500m depth. Its full dip is 10° in direction N 45° E from A. It is intended to sink another shaft B at a distance of, 2000m from A in the direction S60° E. Difference of surface level between A and B is 30m. 'B' being at higher level. At what depth from the surface will the seam be met in 'B' shaft ? **(Survey Comp. 1975)** **(Ans.** 621.28m)

13. A dipping cross-measure drift was at a gradient of 1 in 2 on the line of full dip to recover a coal seam dipping 1 in 5 which has been thrown down by a fault. The roof of the seam was met by the floor of the cross-measure drift at a distance of 181m and the floor of the coal seam at 201m. Calculate the thickness of the seam and check your answer by plotting section to a scale of 1 in 100. **(Survey Comp. 1975)** **(Ans.** 5.27m)

14. A coal seam has a gradient of 1 in 7 in direction due west. At a point B, 1000m due West of the outcrop A, a shaft is to be sunk to the seam. Calculate the depth of the shaft. **(Ans.** 142.857m)

15. A seam of coal outcrops in a level country which dips at 12°. What will be the depth of a pit to touch the coal seam on the line of full dip 1100m from the outcrop ? **(Ans.** 233.81m)

16. Two coal seams lie 50m apart vertically from the floor of the upper seam to the roof of the lower seam. The rate of dip is 1 in 7.5. What will be the length of a level drift between these seams ? **(Ans.** 375m)

17. A coal seam has been bored through and shows a vertical thickness of 4m. The strata dip at 1 in 3. Find the true thickness of the coal seam. **(Ans.** 3.795m)

18. A drift rising 1 in 6 cuts a seam of coal dipping inbye at 10°. The width of the seam as measured along the floor of the drift is 8.5m. Find the true thickness of the seam. (**Ans.** 2.832m)

19. The beds in a mining property dip 18° in the direction due South, and a coal seam having an apparent thickness of 6.5m is passed through by a borehole which is found to be deflected 10° from the vertical where the borehole passes through the seam. Calculate the thickness of the seam, when the borehole is deflected in the same direction as the dip of the beds. (**Ans.** 5.74m)

20. What is the length of a drift, dipping at 1 in 5, between two seams 39 metres vertically apart from floor to floor, assuming the seams to be dipping at 1 in 8 (a) in the opposite direction to the drift, and (b) In the same direction as the drift ? (**Ans.** 122.4m; 530.4m)

21. Two seams of coal have a full dip of 1 in 4 in the direction of N 30° E and they are 30 metres apart vertically. Find by construction (a) the dip of the seams in the direction N 60° E, and (b) the length of a drift rising 1 in 4 in the direction of N 60° E, to connect the seams. (**Ans.** 1 in 4.619; 66.287m)

22. Two seams are situated in the strata 45m apart measured at right angles to the line of stratification. The measure dip due South at 1 in 6, and it is proposed to drive a cross-measure drift rising at 1 in 3 in a bearing of S 60° W from the lower seam to intersect the upper seam.
Plot a section to scale of 1 cm = 25m, and ascertain the length of the drift from seam floor to seam floor. (**Ans.** 114m)

23. The floor of a seam of limestone is cut by a vertical bore at a depth of 67.5m from the surface. The core of limestone taken from the bore is 6m long, and shows that the strata dip at 33°. If the surface rises uniformly at 1 in 12 in the same direction as the strata, calculate : (a) the breadth of the seam at the outcrop measured on the slope of the rockhead, assuming that a thickness of 3m of alluvium overlies the rockhead; and(b) the thickness of the limestone seam measured at right angles to the plane of stratification. (**Ans.** 10.65m, 5.04m)

24. A vertical bore cuts the base of a stratum 4.5m in thickness at a depth of 40.5 metres. The bore core shows the dip of the strata to be 30°. The surface rises at a uniform grade of 1 in 7 in the same direction of the strata. There is an alluvial deposit, 3m in vertical thickness, on rockhead. Calculate (a) the true thickness of the stratum, (b) the slope distance (measured on the slope of the surface) from the bore to the outcrop line of the top of the stratum, and (c) the breadth of the outcrop of the stratum.
(**Ans.** 3.897m; 76.737m; 10.464m)

25. To prove a coal seam three boreholes as under have been put down :

Station	R. L. (above Sea Level)	Depth of hole at which coal is touched	Distance in metres
A	135m	60m	AB = 300m
B	112.50m	75m	AC = 405m
C	120m	90m	

The bearing of BA is N 45° E and ∠BAC = 80°. Thickness of seam in borehole is 4.8 metres. Calculate the rate and direction of true dip and also the true thickness of seam.
(Ans. 1 in 6.473; S 9° 01'W; 4.719m)

26. Bench marks A, B and C in the workings of a seam are situated as follows :
B is 330m from A in a direction N 15° E
C is 330m from B in a direction S 85° E.
Level of Bench marks above an assumed datum 300m below ordnance datum are A =2475m, B = 2482.5m and C = 2467.5m. Calculate the direction and rate of true dip.
(Ans. S 52° 28'E; 1 in 16.86)

27. Three boreholes X, Y, Z have passed a coal seam at depth of 180m, 90m and 60m respectively. A line from X to Y takes a direction N 30° E and measures 300m. A line joining X to Z is in direction S 40° E measures 480m. Calculate the rate and direction of full dip of seam. **(Ans.** 1 in 2.087; S 81° 21'W)

28. Find the direction and amount of true dip of a seam which has been proved by three boreholes X, Y and Z.

Borehole	Distance.	Surface level	Depth of seam
X		20m	35m
	500m		
Y		15m	20m
	500m		
Z		30m	25m

The bearing of XY is N 30° E, that of YZ is S 30° E. **(Ans.** 270°; 1 in 25)

29. Three boreholes A, B and C have been put down in a colliery leasehold to prove the presence of a coal seam. The surface levels of the boreholes above datum are 22m, 14m, and 30m, and the depth of boreholes are 38m, 20m and 26m respectively. The borehole B is 480m away from A on a line bearing N 30° E and borehle C is 480m from B on a line S 30° E. Calculate the azimuth of the line of full dip and also the rate of inclination. **(Ans.** 270°; 1 in 24)

30. Three boreholes A, B and C intersect a seam of coal at depths 180, 208 and 380 metres respectively. A is 600 metres north of C and 800 metres east of B; and B is 1000 metres from C. Determine the direction and amount of full dip of the coal seam.
(Ans. S 7° 58'W; 1 in 3.96)

31. Three boreholes are put down to prove a seam of coal. A line from A to B takes a

direction N 24° E, and the distance is 300 metres. A line joining A and C makes a right angle with line AB i.e. it lies in a direction S 66° E. The distance AC is 375 metres. The depths to the seam in A, B and C respectively are 180m, 90m and 67.5m. What is the bearing of the line of maximum dip ? **(Ans.** S 69° W)

32. Let A, B and C be three boreholes in a perfectly level district. The same seam of coal 0.9m thick has been proved by A at 216m, by B at 360m, and by C at 270m. A is 450 metres from B, and due north of it, and 360 metres from C, which is east of the line BA. The angle between AB and AC is 45°. Find the direction and amount of the dip of the coal. **(Ans.** S 18° 36'W; 1 in 2.96)

33. In a dip heading 1 in 8 an upthrow fault of 12m is met with. What length of drift rising 1 in 6 will be required to seat the coal ? **(Ans.** 41.14m)

34. A road driven to the full dip in a coal seam of 1 in 12 meets with an 80m downthrow fault hading 82° to the horizontal. An incline is set away at the same bearing at 1 in 4. At what distance will it strike the seam on the downthrow side of the fault ?

(Ans. 488.79m)

35. Two coal seams having a dip of 1 in 8 are separated by 40m of strata, measured vertically. A level drift is to be driven from the lower seam to the upper seam. After proceeding ashort distance, a downthrow fault is encountered, causing a vertical displacement of the strata for a distance of 10m. Calculate the length of the drift. **(Ans.** 240m)

36. A road on the strike of a coal seam meets a fault 10m up running on the dip and rise, with a hade of 60°. Draw a section and find the starting point, and length, of a stone drift rising 1 in 10 in the line of the road to meet the seam on the upper side of, and at the fault. **(Ans.** Starting point = 82.66m back from the fault; Length = 100.5m)

37. A roadway in a level seam advancing due north meets a normal fault with a hade of 30° from the vertical and bearing at right angles to the road. An exploration drift is driven through the fault due north and rising 1 in 1. It meets the seam on the other side of the fault after going 30 metres. Find the throw of the fault and the width of the barren ground. If the drift were driven at 1 in 3 instead of 1 in 1, what would be its length ?

(Ans. 21.2m; 10.6m; 67m)

38. A roadway being driven along the dip of the seam having a gradient 1 in 8 meets a downthrow fault. The fault hades at 1 horizontal 2 vertical and the vertical throw is 30m. Calculate -

(i) The length of a cross-measure drift dipping at 1 in 4 in the line of the roadway to win the seam at a point 20m (plan measure) on downthrow side of the fault.

(ii) The slope distance from the rise side of the fault at which the drift must be set off. Assume the coal measures to dip uniformly on both sides of the fault.

(Ans. (i) 232m (ii) 191.48m)

39. The main dip gallery of a seam dipping at 1 in 5 has touched a vertical down-throw fault running in strike direction at point A. A level drift from A has been driven in true dip

direction to a point B beyond the fault, the distance AB being 10 metres. A staple shaft shunk at B has touched the same seam at a depth of 20 metres. Assuming the direction and rate of dip of the seam to be the same on both sides of the fault, find out by calculation the throw of the fault and check by plotting. **(Ans.** 18m)

40. A roadway dipping 1 in 10 in the direction of full dip of a seam strikes an upthrow fault bearing at right angles thereto with a throw of 25m measured from floor to floor along the fault plane with the hade of 30° from the vertical. Calculate the length of the drift to win the seam commencing at the lower side of the fault and rising 1 in 3 in the same direction as the roadway. **(Ans.** 52.85m)

41. A roadway driven on full dip at 1 on 6 meets a normal upthrow fault at B bearing at right angles to the roadway. The hade of the fault is 20° to the vertical. A cross-measure drift is driven at 1 in 4 rising on the same bearing as the roadway and recovers the seam at C, 120m from B when measured along the floor of the drift. Calculate the throw of the fault. **(Survey Comp. 1971) (Ans.** 45.73m)

42. A roadway dipping at 1 in 7 meets a 30m downthrow fault with a hade of 2 vertical to one horizontal. A drift at 1 in 4 is to be driven on the same bearing as the roadway but started some distance before meeting the fault, so as to catch the seam again at point 50m (plan distance) from the dip side of the fault. Calculate the total length of the drift. Assume the gradient of the seam to be uniform and the line of fault at right angles to the roadway. **(Survey Comp. 1972) (Ans.** 267.5m)

43. A level driven in a coal seam dipping at 1 in 4 in direction S 10° W touches a downthrow fault of 40m vertical displacement with a hade of 30°. Strike of fault is N 20° W. Draw a plan and section to show the length of drift driven from A (upthrow side) in a direction S 70° W at a gradient of 1 in 2 to touch the seam on downthrow side. **(Ans.** 110.65m)

44. A seam dipping at 1 in 4 in the direction S 30° W encounters a downthrow normal fault which runs in East West direction. The vertical displacement of the fault is 50m and it hades at 30°. On the downthrow side of the fault the seam dips at 1 in 6 in the direction S 45° W. A stone drift dipping at 1 in 2 in the direction due South is to be driven from the upthrow side of the fault from a point 24m from the fault to touch the seam on the downthrow side. Calculate the length of the stone drift. **(Ans.** 143.23 metres)

45. Shaft sinking has been completed to a new seam. Describe how you would determine the direction and rate of full dip of seam and give a list of equipment required to do this. Illustrate your answer with sketches.

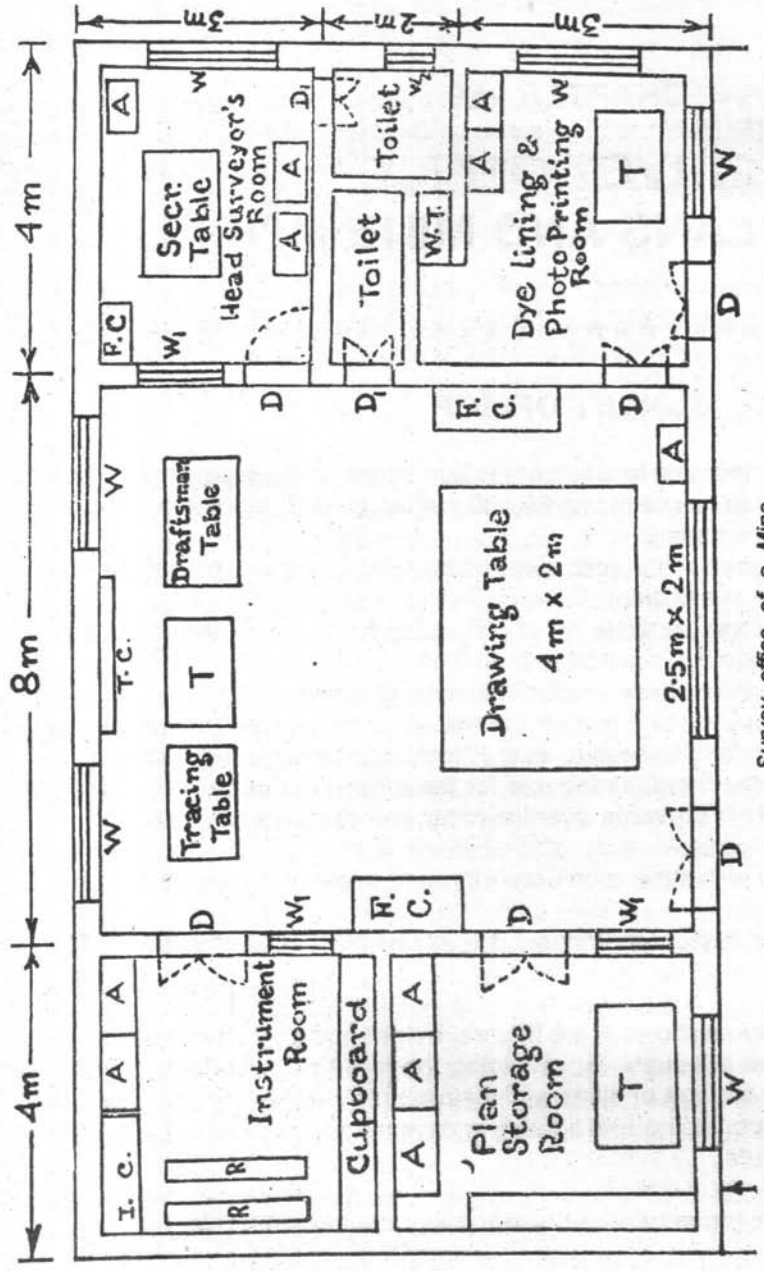
46. A seam dipping at 12° degrees in a direction of S 40° E is intersected by a normal upthrow fault bearing S 50°W, the hade of the fault is 30 degrees from vertical. A cross-measure drift starting from the plane of the fault in the seam 284m above datum and driven on line of full dip strikes the seam beyond the fault at A 352m above datum. The point A is 60m from the fault plane measured on the dip direction of the seam, which also dips at 12 degrees, Calculate the throw of the fault, the inclination of drift and its length. **(Survey Comp. 1992) (Ans.** 80.475m; 1 in 1.546; 125.223m)

CHAPTER - IX

SURVEY OFFICE
SCALES, PLANS AND MINE MODELS

★★★

3. Laquered tin cases for storing old plans.
4. Filing cabinets.
5. Steel Almirahs for storing records.



Survey office of a Mine

6. Instrument cupboard.
7. Drawing tables, and copying table with glass top which can be illuminated from below.

8. Adequate numbers of tables, chair, tools, fluorescent lamps and angle poise lamps.

Instrument Required : (1) Theodolite preferably microptic type with telescopic tripod legs. (2) Vernier dial preferably microptic type. (3) Tilting level - 1. (4) Dumpy level - 1. (5) Trough compass - 1 . (6) Tubular compass - 1. (7) Levelling staff - 4. (8) Ranging rod - 8 (9) Chain - 2. (10) Straright edge (wooden) - 1. (11) Spirit level - 2. (12) Steel tape of verying lengths. (13) Invar tape 30m long. (14) Steel band 30m. (15) Linen or metallic tape 15m and 30m. (16) Other accessories include - Binoculars, pegs, hammers, arrows, plumb-bobs, cross-staff, sighting signals, zinc plates, nails, wires, cotton thread, red oxide, ferro-printing machine, calculator.

Drawing Instruments : The following is a list of drawing instruments which should be found in the survey office -

Celluloid set squares 60^0 and 45^0 of varying sizes; Calluloid circular protractors of varying diameters; French curves; Set of Drawing instruments; Roller type parallel rular; Planimeter; Pantagraph; Beam Compass; Magnifying reader; Diagonal scale; Scales with offsets for statutory plans; Paper board scale; letter stencil plates; Drawing board and T-square; Board pins; Erasers soft and hard; Pencil of varying hardness; Colour brushes; Drawing pens; Bottle of black Indian ink and chinese ink; Coloured inks; Porcelain palettes; Sufficient quantity of Mounted paper; Tracing cloth, Tracing paper, Graph paper, White paper, Drawing sheets, Carbon paper and paper for ferroprints; Paper weight; Colour box - cakes and tubes, French chalk, Sharp knife; Blade; Ordinary pins; Jems clip and gum paste.

Personnel Required : The following personnels are required for the survey office in a mine producing 30,000 tonnes per month -

Head Surveyor - 1. Surveyors - 3. Assistant Surveyor - 2. Chainman - 6. Draftsman - 1. Tracer - 1. Ferro printer - 1. Clerk - 1.

Storage of Survey Instruments : The following arrangements should be made for the storing of surveying instruments to keep them in good condition -

All the survey instruments after use and before storing should be wiped clean and dry, and the metal portions rubbed with a piece of chamois leather and a very little watch oil. Magnetic needle should be unclamped and the instruments properly placed when returned to their boxes or cases, care being taken that no undue force is applied. Instruments should be kept on the cupboards specially made for the purpose.

Ranging rods should be formed into bundles and laid flat on the floor or placed perfectly upright in suitable racks or niches. For the entire protection of a levelling staff a leather bound sail-cloth should be used. This may be divided into two compartments for the staff and tripod with pads between which forms a protection from slight bruises and dirt when set up slanting in a corner for furture use.

The store room and lockers should be in dry position, of fairly equable temperature. A list of all instruments with their respective types, specification and identification numbers should be maintained.

Storage of plans : The following arrangements should be made for proper storage of mine plans -

1. Drawers and cupboards fitted below the largest table for flat storage of main surface and underground plans. Where the size of the plan is such that a large enough drawer or cabinet is not available, it should be sub-divided into a number of cabinet-sized section each being carefully marked and indexed to correspond with the sections enjoining.
2. Separate plan lockers and drawers for other plans, suitably indexed to indicate plans contained therein.
3. Cylindrical lacquered tin cases for storing old plans on wall racks.
4. When plans are to be stored in rolls, these should have a roller fixed to one end, and the opposite edge should be protected with silk or leatheer edging to prevent it from becoming damaged or broken.
5. All plans shall be serially numbered and a list shall be maintained in a bound-paged book kept for the purpose.

Minimising and Detecting Shrinkage : The paper which is to be used in the production of mine plans should be tough, opeque and well-seasoned and it should be suitably mounted. It should be allowed to remain in the office flat and untouched for at least six months to acclimatise it to the existing temperature and humidity.

In large plans the shrinkage may not be uniform, being greater at some parts than at others. When sufficient time has been allowed for the proper seasoning of the paper, and before any plotting has been done, the paper should be divided into suitable squares or co-ordinate network, so that it may then be possible to determine the amount of variation in the area of any of the squares due to expansion or contraction, and the correction to be applied to subsequent plottings.

SCALES AND ITS CLASSIFICATION

Classification : Scales may be classified into the following types :

1. **Simple Scale or Plain Scale :** This shows equal primary (major) and secondary (minor) divisions only. Plain scales are suitable for coarse graduations and are easy to read.

2. **Diagonal Scale :** This is used for obtaining very minute subdivisions when such divisions can not be obtained from plain scales.

3. **Vernier Scale :** It is a small auxilliary scale which slides along the main or primary scale and is so graduated that its divisions are a definite fraction either less, or greater, than the primary divisions. The vernier carries an index point or arrow, the position of which is to be recorded and which forms the zero of the vernier divisions.

4. **Comparative Scale :** The scales which have a common representative fraction but which read in different measures are called comparative scales. Measurements can be taken directly of the plan in the desired units by means of a comparative scale and no calculation will be required.

5. Proportional Scale : This is used for enlarging or reducing a drawing to given proportion.

Characteristics of a Good Scale : The characteritics of a good scale are -

(1) It should read to the greatest accuracy.
(2) It should be convenient in use.
(3) It should be accurately divided and clearly numbered.
(4) Its dividions should be finest.
(5) It should be sufficiently long, but usually not more than 30cm.
(6) It should be easily read and no arithmetical calculations should involve in measuring distances on the plan with it.
(7) The zero division must always be placed between the unit and its subdivisions on the left.
(8) The name of the scale together with its representative fraction should be written on the plan.

VERNIER SCALES

Vernier : The vernier is a device which enables measurements to be taken to a finer degree of accuracy that would be possible by direct estimation of the divisions on the main scale and it is particularly useful when the further subdivision of this scale would give graduations closely engraved to be easily distinguised. It consists of a small scale which moves with its graduated edge in conjunction with the graduated edge of the main scale.

Principle of the Vernier : It is a fact that the eye can readily detect any abrupt break in the continuity of a straight line. Usually the vernier and primary scale graduations are out of continuity except at one point. In reading a vernier the eye is passed along it until two lines are found, one on the primary scale and the other on the vernier scale, which are exactly coincident. The value of this line as given on the vernier scale is then taken as the vernier reading.

The principle of a vernier is that if 'n' divisions on the vernier cover exactly $(n \pm 1)$ divisions on the main scale, then step by step, the one scale will at each step gain, or lag behind, the other by $\frac{1}{n}$ th of a scale division.

Let, minor division on the main scale = a
Length of graduated space on the vernier = b
number of divisions on the main scale = n

Now, take (n + 1) divisions from the main scale and divide it into n equal parts for the vernier, then each latter subdivision will be greater than the former by the $\frac{1}{n}$ th part of a subdivision on the main scale.

$$\therefore \quad (n+1)\,a = nb \qquad \text{or,} \quad na + a = nb;$$

$$\text{and,} \quad b = a + \frac{a}{n} \qquad \text{or,} \quad b - a = \frac{a}{n} \times a$$

Again, if instead of (n + 1) divisions, (n – 1) divisions are taken from the main scale and divided into n equal parts, then

$$(n-1)\,a = nb \qquad \text{or,} \quad na - a = nb;$$

$$\text{and,} \quad b = a - \frac{a}{n} \qquad \text{or,} \quad a - b = \frac{1}{n} \times a.$$

Various Types of Vernier Scales

(a) Direct Vernier : Verniers in which the smallest vernier division or subsibiary scale division is less than the smallest main scale division, are known as direct verniers or forward reading verniers. In the direct vernier both scales are graduated in the same direction, i.e. both from left to right or both vice versa, so that if the zero of the vernier coincides with the zero of the main scale, the first division on the vernier falls short of the first main scale gradation by $\frac{1}{n}$ th of the main scale division and nth vernier division will coincide with the (n – 1) th graduation on the main scale. These verniers are simpler to read.

(b) Retrograde Verniers : Those verniers in which the least vernier scale division or subsidiary scale division is greater than the main scale division, are called retrograde or backward reading verniers. The scales are graduated in opposite directions, one from right to left and the other from left to right. The advantage of a retrograde vernier is the graduations are not quite as fine as those of direct vernier because the vernier divisions are longer than the main scale divisions in this case.

(c) Double Verniers : With a simple vernier - either, direct or retrograde readings can be taken in one direction only. This may be avoided by placing two simple verniers end to end, forming one scale, with the zero in the centre, and using one of these for clockwise motion and the other for counter - clockwise. The combination, which is double the length of an ordinary vernier, is known as double vernier.

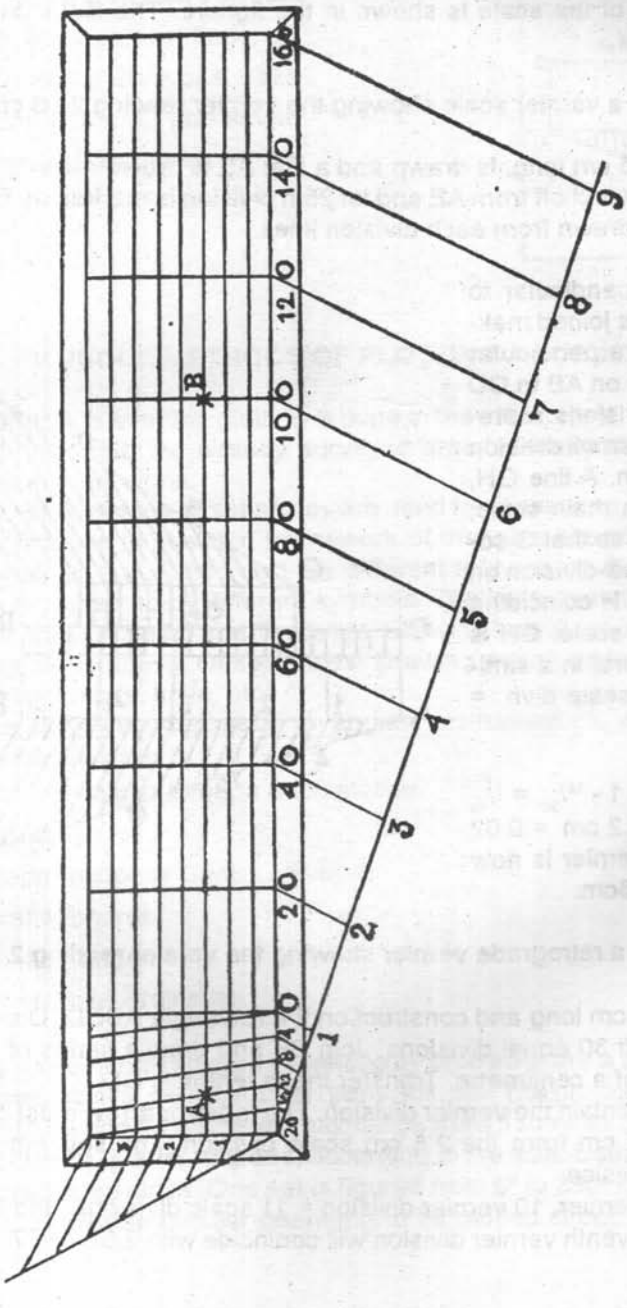
(d) Extended Verniers : This type of vernier is used when the available length of the vernier is small, and when it is required to have a small least count without having vernier divisions too close together for convenience in reading.

(e) Arc Verniers : The verniers embodied in mine surveying instruments are necessarily circular in shape, the length of the arc of the vernier being dependent on the accuracy with which the readings of angles should be given by the vernier. Direct verniers are invariably fitted to mine-surveying instruments : the index or arrow may be placed at one end as with a straight vernier, or in the middle, to economise space. The latter arrangement is called a

folded vernier. On mining dials, the verniers may read to either 3 minutes or 1 minute and on theodolites, they read upto 20 seconds, or, exceptionally, 10 seconds.

CONSTRUCTION OF SCALES

Example 1. Construct a scale of 1 : 1200 showing the reading 113.5m and reading to 50cms. (1cm = 0.3937 inch).



Diagonal Scale R. F. 1 : 1200
Showing Metres and 50 cms.
AB = 113.5 Metres

Solution : A line 15 cms long is drawn. 15cms on scale representing 15 x 1200 = 18,000 cms or 180 metres in actual measurement.

The line is divided into 9 equal parts each representing $^{180}/_{9}$ = 20 metres. One part (20m) is then sub-divided into 5 equal parts, thus each sub-division representing $^{20}/_{5}$ = 4 metres.

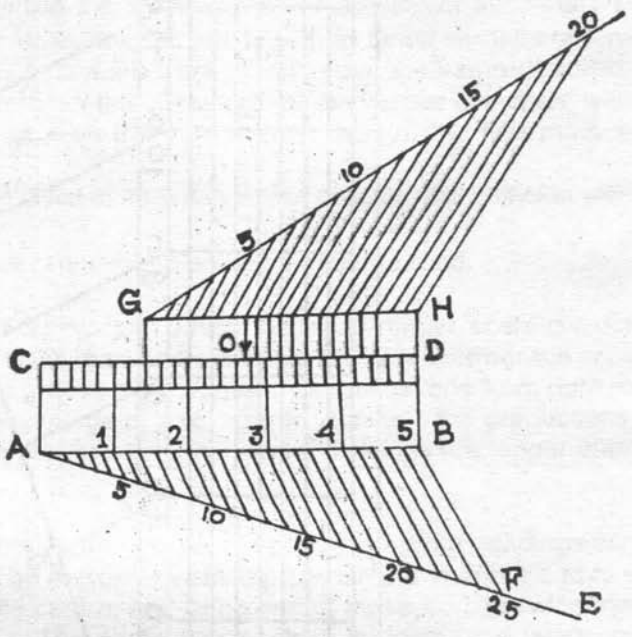
To get the smallest reading 50 cms, one 4m division is divided into $^{400}/_{50}$ = 8 divisions diagonally.

The construction of the scale is shown in the fighure. The distance AB in the figure represents 113.5 metres.

Example 2. Construct a vernier scale showing the vernier reading 2.66 cm.

Solution : A line AB, 5 cm long, is drawn and a line AE is drawn making an oblique angle with it. 25 equal parts are cut off from AE and let 25th division is marked as F. BF is joined and lines parallel to BF are drawn from each division lines.

AC and BD, perpendicular to AB, are drawn and CD is joined making it parallel to AB. Perpendicular lines from each division on AB to CD are drawn. Every 5 divisions representing 1 cm and every small division on main scale is 0.2 cm. A line GH, 18 subdivisions long on main scale, is drawn parallel to CD, so that G coinciding with 1.4cm (7 sub-division on main scale from C) and H coinciding with 5cm at D on main scale. GH is divided into 20 equal parts in a similar way. Thus 18 main scale divn = 20 vernier division.



$\therefore$ Least count = 1 - $^{18}/_{20}$ = $^{1}/_{10}$ of main scale = $^{1}/_{10}$ x 0.2 cm = 0.02 cms. The zero of the vernier is now indicating a reading 2.66cm.

Example 3. Construct a retrograde vernier showing the vernier reading 2.57cm.

Solution : Draw AB 3 cm long and construct on it a rectangle ABCD. Draw AF at an acute angle to AB and step off 30 equal divisions. Join BF and draw a series of parallel lines, so dividing AB into tenths of a centimetre. Transfer these tenths to CD.

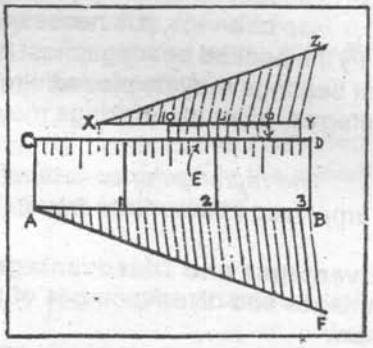
Now draw XY to contain the vernier division. The index or arrow must be between 2.5 to 2.6cm and exactly 0.07 cm from the 2.5 cm scale division. The seventh division E, must coincide with a scale division.

On a retrograde vernier, 10 vernier division = 11 scale divisions, and 7 vernier division = 0.77 cms. Thus the seventh vernier division will conincide with 2.57 - 0.77 = 1.8 cms on the

main scale. For construction purpose, it is desirable to show 20 vernier divisions and to select the appropriate 10 divisions from then.

If the vernier be extended to the left from E, its tenth division X from E will coincide with 1.8 – 1.1 = 0.7 cm on the main scale and if it be extended to the right, its tenth division Y from E will coincide with 1.8 + 1.1 = 2.9cm on the main scale.

From X, draw XZ and step off 20 equal parts. Join ZY and draw parallel lines, so dividing XY into 20 vernier divisions. Take three divisions to the left of E, and seven divisions to the right of E and finish off the vernier as shown.



VARIOUS METHODS OF PLOTTING A SURVEY

After the field work is over the plotting is done in the office. Plotting means representing on paper to a suitable scale the already surveyed area. It consists in drawing plans and sections using conventional signs.

For plotting a chain survey offset scales are used for measuring offsets on both left and right hand of the chain line in skeleton framework of triangles already plotted covering the area. The various offsets are plotted and the different features on the ground and also the boundary lines are represented by different symbols. The instruments required for plotting a chain survey are : A drawing board and T- square, Drawing box, A set of set-squares, French-curves, Scales, Trough compass, Offset scales, Drawing paper, and pencil, eraser, drawing pins, colour box, ink, brushes, knife, etc.

For plotting a survey conducted by an angular instrument (by dial or theodolite, etc.), the following methods are used :

1. By parallel meridians through each station.
2. By included angles.
3. By paper protractor or Centre meridian.
4. By method of chords.
5. By method of tangents.
6. By rectangular co-ordinates.

The drawing instruments required in *plotting by protractor* are a circular protractor, parallel ruler, set squares, scales, pencil, rubber, pens, ink, etc. The protractor used for plotting is usually circular in shape and its diameter may range from 15cms to 60cms. The circumference is divided into half or quarter degrees, according to the size. Usually, two sets of figures are shown on the graduated circle. One set is figured from 0° to 360° in a clockwise direction to enable azimuths (clockwise circular bearings) to be plotted direct. The other set may be

figured from 0° to 360° in an ani-clockwise direction; or it may be figured from 0° to 90° in each quadrant, the zeros coinciding with the 360° and 180° of the whole-circle figures.

If the bearings booked during a survey are azimuths, it is necessary to use the clockwise graduations on the protractor for plotting. Conversely, if the bearings booked are anti-clockwise bearings, it is necessary to use the anti-clockwise graduations for plotting; alternatively the booked bearings must be subtracted from 360° to convert them into azimuths. Quadrant bearings may be plotted direct from the quadrant graduations on the protractor; whilst, if preferred, azimuth bookings may be converted into quadrant bearings for plotting purposes.

The rectangular co-ordinates method is used in plotting very precise work. Generally it is employed in theodolite traversing.

Advantages and Disadvantages of plotting by co-ordinates and protractor : The advantages and disadvantages of co-ordinate plotting over plotting by protractor are tabulated below -

Plotting by Co-ordinates	Plotting by Protractor
1. Distances and bearings can be calculated as same degree of accuracy as that of field measurement.	1. Distances and angles taken off a plan plotted by protractor is limited to the scale of the plan, the smallest divisions on the protractor and the skill of surveyor.
2. Each point is plotted independently with reference to meridian and at right angle to it and not with reference to the preceeding point. Thus if any point is wrongly plotted others are not affected thereby, and errors can not accumulate.	2. If one draft is misplaced all the succeeding drafts will also be plotted wrongly and a cumulative error in plotting occurs.
3. The position of each point can be checked by mere scaling the distance between two points and comparing it with field measurement.	3. It lacks this advantage.
4. Shrinkage of plan at any part can be ascertained readily from co-ordinate graticules or squares which cover the plan with a network of scale.	4. Shrinkage can not be readily ascertained.
5. In a closed traverse accuracy of work can be determined before plotting. Closing error can be distributed by giving due weights to measurements if necessary.	5. Closing error can not be discovered before plotting and if any error is detected it can not be known to what extent error is due to plotting or worng field work.
6. The computation form is a record of the positions of each station independent of the plan and is available for subsequent checking.	6. For subsequent checking the whole survey is to be replotted.

Plotting by Co-ordinates	Plotting by Protractor
7. Areas of closed figures may be calculated directly from co-ordinates.	7. Areas of a closed figure may be found by equalising the plotted lines to form a triangle and by measuring the base, height etc. which is troublesome and accurate results may not be expected.
8. Before plotting, the origin of co-ordinates may be selected from the co-ordinate position of points and the survey can be accomodated centrally on the sheet.	8. The position of points can not be selected beforehand to accomodate the survey lines within the limit of the sheet.
9. Plotting of large survey can proceed simultaneously on different sheets or on different parts of same sheet.	9. This is not possible in protractor plotting.
10. The method is labourious as it involves lengthy calculations, squaring of paper accurately.	10. It furnishes the most rapid method of plotting.

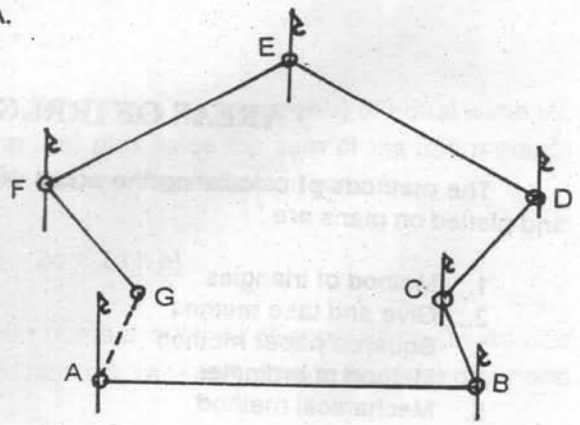
Example 4. Plot the following notes on a scale of 1cm = 10m.

Line	*Bearing*	*length*
AB	Due East	80m
BC	N 30°W	20m
CD	N 45°E	30m
DE	N 60°W	60m
EF	S 60°W	60m
FG	S 45°E	30m

Find the length and bearing of line GA.

Solution : The survey is plotted to a scale of 1 cm = 10m as shown by ABCDEFG.

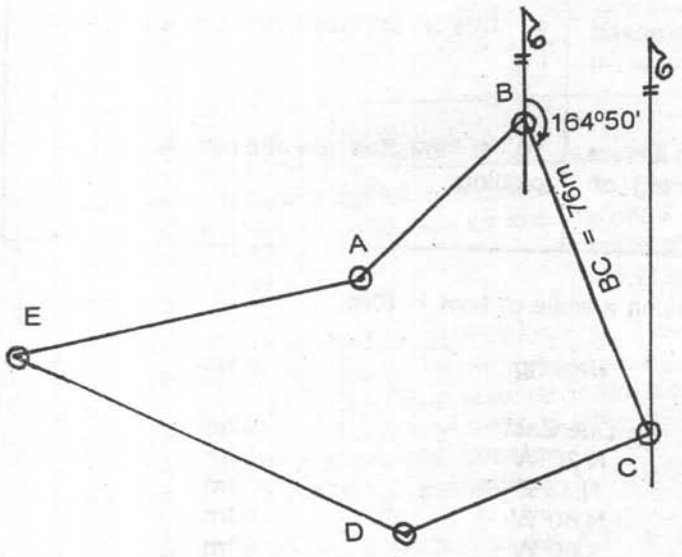
Length of GA = 20m
Bearing of GA = S 29°30'W



Example 5. The following are the notes of a closed traverse :

Line	*Azimuth*	*Distance*
AB	40°30'	43m
BC	closing line	-
CD	250°00'	47m
DE	290°00'	78m
EA	65°00'	75m

Plot the traverse on a suitable scale and ascertain by measurement the azimuth and length of BC.



Scale 1 cm = 2m

Solution :
The survey is plotted by protractor to a scale 1 cm = 20 metres.

Measured bearing of BC = 164°50'

Measured length of BC = 76m

AREAS OF IRREGULAR FIGURES

The methods of calculating the areas of irregular ground which have been surveyed and plotted on plans are :

1. Method of triangles
2. Give and take method
3. Squared paper method
4. Method of ordinates
5. Mechanical method

1. Method of Triangles : (a) In the case of a figure whose outline consists of a series of straight lines, the figure is to be converted into a single triangle of same area by equalising on the principle that triangles upon the same base and between the same parallel lines are equal. The base and perpendicular height are scaled off from the plotted survey unless measured beforehand.

$$\textit{Area of triangle} = {}^1/_2 \text{ (base x height)}$$

(b) Where the sides of a triangle a, b and c are the measured lengths obtained from the field book of a traverse survey or from a trigonometric triangulation survey,

$$\textit{Area of triangle} = \sqrt{s\,(s-a)\,(s-b)\,(s-c)} \quad \text{Where } s = {}^1/_2\,(a+b+c)$$

(c) When two sides and included angles of a triangle are given, the *Area* = $^1/_2$ (ab Sin C).

Here, a and b are two sides and C is the included angle.

(d) When one side and the adjacent angles of a triangle are given,

$$\textit{Area} = a^2 \text{ Sin B Sin } \frac{C}{2} \text{ Sin (B + C)}$$

2. Give and Take Method : The areas of figures with irregular outlines can be obtained with accuracy by dividing them into triangles, preceded by "equalising", which consists in drawing "give and take lines" through the curved boundaries.

3. Squared Paper Method : Small irregular areas may be obtained from a drawing by laying upon it a transparent sectional paper ruled into small squares, each representing a given area. By counting the whole squares, and then adding the fractions of squares the area of the figure can be obtained.

4. Method of Ordinates

Mid-ordinate Rule : Divide the figure by parallel lines into any number of strips of equal width and erect mid-ordinates. Add the lengths of the mid-ordinates together and divide by the number of mid-ordinates erected, to find the mean value m. Then

$$\textit{Area} = \text{length of figure x m.}$$

Trapezoidal Rule : Divide the figure into a number of strips (or trapezoids) of equal width W. Multiply the sum of the extreme ordinates (a and e) plus twice the sum of the intermediate ordinates (b, c and d) by half the common width between two adjacent ordinates.

$$\textit{Area} = \frac{W}{2}\,(a + 2b + 2c + 2d + e)$$

Simpson's Rule : Divide the figure into an even number of strips of equal width (by an odd number of ordinates). Add together the first and last ordinates, twice the sum of the other odd

ordinates, and four times the sum of the even ordinates, and multiply the result by one-third the common distance 'W' between the ordinates.

$$Area = \frac{W}{3} (a + 4b + 2c + 4d + e)$$

Rectangular co-ordinates : The area of any closed traverse can be calculated from the field data. It does not necessitate the preparation of a plan.

5. Mechanical Methods

Mechanical methods are now largely employed for determining the areas of figures obtained in surveying practice. These may be -

(a) By Talc square.
(b) By Area-combs.
(c) By computing scale or computer.
(d) By planimeter.

The use of planimeter is the best and expeditious method and the results are far more accurate except by direct calculation from the field notes. Planimeters are made in many different forms, but in every case a pointer, generally called a tracer, is guided round the border of the area, causing motion in the mechanism which results in the rotation of a wheel; from the amount of this rotation the area of the curve is determined. Different types of planimeters are : (i) Fixed type, (ii) Proportional type, (iii) Rolling sphere type. (iv) Precision disc polar type, (v) Rolling disc type, (vi) Carriage type, (vii) Rule type, (viii) Hatchet type. (ix) Integrator or Moment type.

PLANIMETER

The planimeter is used for measuring areas of irregular figures plotted to scale. It consists of -

(a) a radius bar,
(b) a tracing bar,
(c) a graduated wheel or drum,
(d) a vernier, and
(e) a dial.

The radius bar and the tracing bar are hinged together and permits an angular motion of nearly 180°. The radius bar is of fixed length and carries a steel needle point or pole which is pressed into the plan paper and thus forms a fulcrum around which the instrument can rotate. The tracing bar is of adjustable in length and carries a tracing point which is made to traverse the boundary of the area to be measured. A small weight above the tracing point serves to keep it in position.

The graduated wheel resting on the plan partly rotates and partly slides over the plan as the tracing point moves. The wheel is connected by wormgearing to a horizontal dial which records the total number of complete revolutions of the wheel and the fraction of a revolution

is indicated on the wheel by reference to the fixed index of the vernier which is placed beside the dial.

The wheel is graduated into 10 divisions and each division is sub-divided into 10 parts each representing one-hundredth of a revolution. The vernier facilitates to read to the nearest thousandth of a revolution. Each complete revolution of the wheel indicates a particular units of area to which the tracing bar is set.

In a normal way, the needle point is placed outside the area to be measured and the tracing point is moved around the perimeter of the figure. The number of revolutions of wheel gives the direct measure of the area enclosed, depending the unit of area to which the instrument is set.

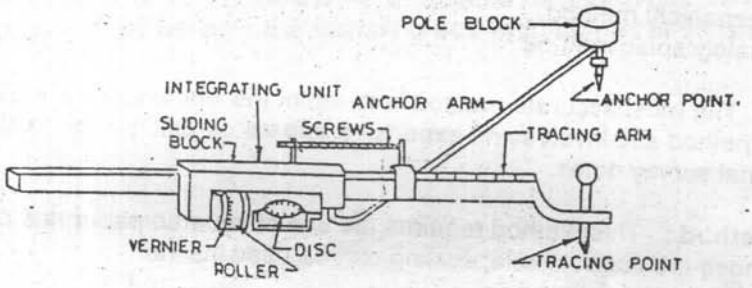
When the needle point is placed inside the figure due account must be taken of the area of the zero-circle, and the instrument reading must either be added to, or subtracted from the area of the zero-circle, according to whether the wheel has rotated in a forward or backward direction.

If the reading is forward :

area = [(Final reading-intial reading) + Zero-circle constant] x unit of area.

If the reading is backward :

Area = [Zero-circle constant - (Initial reading-Final reading)] x unit of area.



Accuracy : With ordinary care, the error in measurement should not exceed about 0.5 percent, being less as the measured area increases.

Precautions : (1) The area should be measured twice with different position of the needle point and the mean area should be taken as correct.

(2) The surface of the plan paper should be smooth. Roughness of the paper, cracked or

uneven surface, a folded plan or a highly glazed paper can never give satisfactory results.

(3) For a too large area, the figure should be divided into parts and each part should be measured separately and the results added to get the whole area.

(4) The tracing point should be guided around the plot by means of a straight-edge or a French curve.

(5) The accuracy of the tracing arm for any given unit should be verified by drawing a square of known size and measuring its area with the instrument.

Zero Circle : If the two arms of a planimeter are arraneged in such a manner that the line joining the needle point and the wheel is maintained at right angles to the tracing bar, the tracing point describes a circle known as the zero circle of the planimeter. In this position the wheel does not revolve but only slides and in spite of the tracing point traversing around a certain area the wheel gives a reading of zero. When the area of a figure is being computed due account must be taken of the area of the zero circle.

METHODS OF ENLARGING AND REDUCTION OF PLANS

It is frequently necessary to enlarge or reduce maps and plans partially or wholly. Maps and plans may be enlarged or reduced by the following methods :

(1) Replotting the whole survey to the required scale.
(2) Graphical method or by method of squares.
(3) Mechanical method.
(4) Photographic method.

Replotting : The most accurate method is to replot the whole survey to the scale required. It is the best method and involves no expensive apparatus. But it is laborious and it necessitates the orginal survey notes.

Graphical Method : This method requires the use of squared paper and proportional compass. But if these are not available, scaling can be used instead.

The procedure is as follows -

(a) On a piece of tracing paper large enough to cover the original plan a series of squares of some difinite size, according to the size of the plan concerned and amount of detail it contains, are marked. The squares or lines are numbered. This piece of tracing paper is now securely weighted or pinned over the original plan.

(b) Another tracing paper or a drawing paper on which the enlarge or reduced copy is to be made is covered with a similar net work of sqaures having their sides larger or smaller, as the case may be in the ratio of the two scales. These squares are indexed in the same manner as previously done.

(c) The intersection of plan features across the squares of the original plan are now transferred on the corresponding squares for reproduction by means of a scale or by proportional compasses or simply by estimation.

(d) When all the salient points are plotted on the copy, the lines connecting these points are joined up to obtain an accurate reproduction of the original as required.

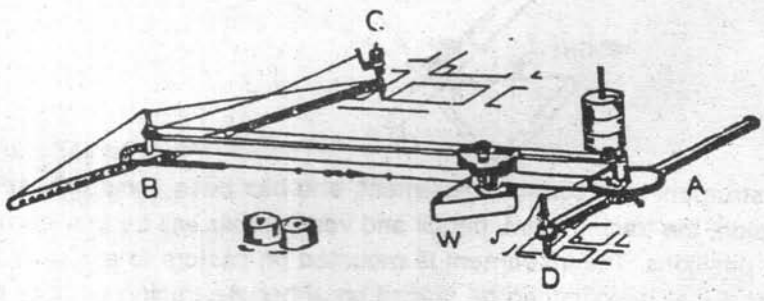
This method ensures accuracy but is more laborious and time taking. It is not quite so satisfactory for enlarging plans, because any error in the plotting of the original plan will be multiplied, according to the scale of enlargement, in the finished plan.

Mechanical Method : This is effected by means of instruments called the Eidograph and the Pantagraph.

The Eidograph : This is an instrument used for mechanically reducing, copying, or enlarging plans in any desired proportion within its range. It can reduce a plan accurately down to about one-eighth of full size, but an enlargement in this or any other ratio is naturally less accurate because any error or deviation of its tracing point is magnified by its pencil point.

The instrument is based on the principle of equal triangles. It consists of three square section brass bars, pivoted at three fulcrums. The fulcrum at the middle is a pin mounted on a heavy weight and forming the point of support for the instrument as a whole. Mounted on the pin, but free to turn in a horizontal plane, is a sleeve or box, with vernier and clamping screws, through which passes the divided centre-bar or beam.

At each end of the centre bar, is a grooved wheel or pulley, the spindle of which revolves in a socket fitted to the end of the bar. An adjustable steel band passes around the two pulleys. Each pulley carries beneath it a clamping box (with vernier and clamping screws)



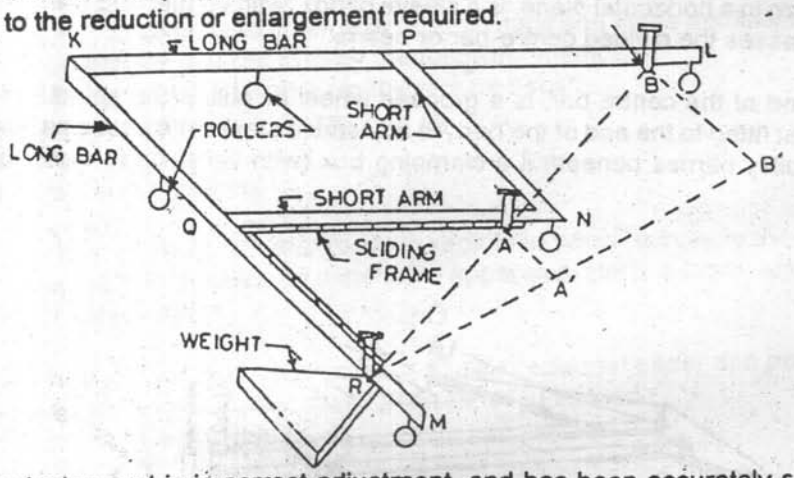
through which slides one of the divided bars, these always being parallel to each other when the instrument is in adjustment.

All these three bars are divided exactly into 200 equal parts which are figured so as to read 100 each way from zero. One bar, called the pencil arm, has a pencil for making the required copy. The bar on the other side is the tracing arm and carries a metal tracer which must be traced by hand over the lines of the plan to be copied.

After the instrument is set, the tracer is passed over the plan to be copied, line by line, and the pencil makes an enlarged, on reduced copy, as the case may be, on a sheet of paper placed in position under it. The copy is reproduced upside down.

The operator may raise the pencil clear of the paper by pulling a chord whenever he desires to move the tracer without its movement being reproduced in the copy.

The Pantagraph : The instrument is based on the principle of similar triangles. It consists of four square section bars, two long bars equal in length hinged at one end and two short bars also equal in length hinged to the mid-points of the long bars and to each other at the other end to form a parallelogram having equal sides. At the extremity of one of the long bars is a tracing point whilst the other long bar passes through a frame carrying an index mark and the vertical axis of rotation. The frame will slide along the bar and the vertical axis of rotation is mounted on a brass weighted base. On one of the shorter bars there is a sliding frame which also has an index mark and a pencil. The frames are clamped to divisions which are marked on their respective bars by the makers. The divisions are marked $^1/_2$, $^1/_3$, $^1/_4$, etc., according to the reduction or enlargement required.



If the instrument is in correct adjustment, and has been accurately set to the desired reduction division, the tracing point, pencil and vertical axis will be in alignment and will remain so in all positions. The instrument is mounted on castors to assist its free movement over the paper. Small weights can be placed on either the tracing point or the pencil to increase the pressure on the paper. A thread passes round a system of small pulleys mounted on the bars, from the tracing point to the pencil so that the operator, on pulling the thread, can lift the pencil if it is required to move the tracing point without marking the copy of the plan.

When using the instrument, it is first set to the ratio representing the degree of reduc-

tion. The original plan is placed in a position which can be covered by the tracing point and the paper for the copy beneath the pencil point. The tracer is then drawn over the detail on the original and the pencil reproduces a copy reduced to the desired scale. The main disadvantage of the pantagraph is that it possesses too many joints and points of support. When used for enlarging plans, any iregularities in the moving parts will be magnified at the pencil, which is interchanged with the tracing point.

Photographic Method : This method is much quicker than the other methods and another advantage is that a large number of small negatives can be stored in a minimum of filing space, to be reproduced to any disired scale as required. The equipment consists of a slide arrangement on which is mounted a camera unit and projector which move along the slide on wheeled carriages or trolleys. At the end of the slide is a screen or easel into which the original plan or sensitized paper can be loaded.

To reduce or enlarge a plan, the first stage is to photograph the plan and process the negative in the usual way. The negative has now to be projected on to sensitized paper to the required scale.

MINE MODELS

Mine models are chiefly to supplement mine maps and elucidate complex vein or fault structure. It may be intended to illustrate mining method, and in such case the scale used is so large that only a part of the mine can be shown. It may also be used for exhibition purposes chiefly for information of laymen, who are not familiar with underground condition.

According to the type of construction and purpose for which they are made, models may be classified under six heads -

1. Skeleton working models.
2. Vein Models.
3. Peg Models.
4. Mine Timbering Models.
5. Soild Glass or Wood Models.
6. Glass Section Models.

1. Skeleton Models : These are small-scale reproductions of shafts, drifts, stopes, etc, based on mine maps, in wood by light iron or wooden trusses in thier true relative positions. Steel may be used for structural supports and for shafts which extend to the base of the model. The principal workings, other than stopes, are made of strong, tough wood and stopes are of light wood for readily cutting into irregular shapes. Level workings are cut in rough,

shaped with knife and fastened together with brads, wooden pins and glue. Each working is given its true gradient. Skeleton models are painted light grey, but contrasting colours, like red, blue and yellow, are used for showing geological features, or deposits and faults. The skeleton supports and trusses are painted black. Names of shafts, veins and working numbers are lettered with water-proof ink or by pasting printed labels.

2. Vein Models : These are made to scale from cross-section and level maps to show vein boundaries. A well-designed vein model may exhibit effects of vein intersections, faulting, and changes in strike and dip. The vein is built of solid, tough wood, beginning with the lowest level. If the vein is irregular the whole may be shaped from a single piece of wood, otherwise the level sections are made seprately and combined afterwards. All the mine workings are outlined with ink on the vein.

3. Peg Models : These are commonly used for tubular deposits like the disseminated coppers proved by vertical drill holes. The surface of a table or board is enameled over which the boundary line and surface geology are drawn to scale and small holes are bored in the table to indicate the corresponding drill holes on the ground. Small pegs are set in these holes. The surface of the board or table represents a certain elevation above mean sea level, and the lengths of pegs are such that the top of each represents the surface reduced level at the corresponding drill hole. To show different formations cut by the drill holes, pegs are painted with different colours.

4. Mine Timber Models : These models are chiely used for exhibition purposes, to show the method of timbering. These models may be made only where square-set method of timbering is used and where accurate stope records are kept.

5. Solid Models :

(a) *Solid glass models* consist of superposed plate-glass sheets, in which are cut openings corresponding to those in mine. Accurate stope plans or sections are required at 2.5m intervals, each glass sheet corresponding to plan or section. Outline of each opening is marked on glass, sawed out as in scroll- sawing, and inner surfaces of openings are painted with any desired colour. Glass sheets are cemented together strongly. These models are costly.

(b) *Solid wood models* are made of wooden sheets, 15mm to 50mm thick, each representing a vertical slice through the mine. The workings and geological structures are drawn on wood with openings cut to correspond with mine workings. The lower edge of each sheet rests on the base and the upper edge shaped to represent the topography of the slice. After assembling, the boundary lines, outcrops, buildings, etc. are drawn on the upper surface. For ready access to any section of the model, the slices may be hung on hinges. Wood, clay plaster of paris. etc. may be used to illustrate the surface features.

6. Glass Section Models : These are the cheapest and most effective for geological study in connection with mine development. These consist of horizontal or vertical sections.

The Horizontal Section Models are made by transferring the mine level workings to glass sheets, so spaced in vertical arrangement as to show the workings in their true vertical and horizontal relations. The workings are outlined on the upper side of the glass by a mixture

of black oil paint with gold size and turpantine. For showing geological features different colours, corresponding to those of the mine maps, are used on underside of the glass. All working lines and geological features are traced directly by placing the glass on the map. Size of the glasses should not be very large on account of excessive weight and danger of breakage. Ordinary window glass of uniform thickness and free from flaws may be used for the construction of glass section models. Electric bulbs may be set on base of these models.

The *Vertical Section Model* supplements the horizontal type, for showing vertical relations of structural features of the deposit. The sheets are spaced according to plans or sections and set in grooves in a wooden frame. This type is very useful for study of deposits having large lateral dimensions, as compared with vertical extent, such as disseminated copper deposits.

Several modifications of glass section models have been made. All models should be of dust-proof construction. Glass sheets may loose transparency because of dust. Complete glass cases for permanent models or loose wooden box cover of light construction for models used at intervals should be used.

CHAPTER - X

OBJECTIVE TYPE QUESTIONS

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OBJECTIVE TYPE QUESTIONS

CHAIN SURVEYING

Fill in the blanks with suitable words

1. A steel tape is.....to a metallic tape.
2. Hypotenusal correction for 20m chain for measuring along slopes is given by.....links.
3. Surveying is the first.....for the execution of any engineering project.
4. Smallest basic unit of length in metric system is..... .
5. The accuracy of a chain survey is..... .
6. Ill conditioned triangles having angles..... .
7. The principle of a chain survey is..... .
8. In an underground roadway stations are fixed in.....for carrying out survey work.
9. The coffcient of thermal expansion for invar tape is..... .
10. The most common instrument for linear measurements are.....and..... .
11. Geiodetic surveying is conducted for areas over..... .
12. In plane surveying knowledge of.....and.....is essential.
13. The composition of invar tape is.....and..... .
14. The geodetic surveying is conducted by..... .
15. The difference between a plan and a map is that the.....of former is.....than the latter.
16. The correction for slope is always..... .
17. The triangulation is the basis of..... surveys.
18. The length of one link of a metric chain is..... .
19. In an optical square the index and horizon glasses are placed at an angle of..... .
20. The use of cross staff is..... .
21. The ranging done between two points which are not intervisible is called..... .

22. In metric chain one metre length is divided into..... .

23. The brass handle of a chain is provided with a.....joint.

24. Incorrect length of a chain gives.....error.

25 The preliminary inspection of the area to be surveyed is known as..... .

26. Stepping method in chaining is used for measuring horizontal distances on .....ground.

27. Test gauge is used for testing..... .

28. A 30 metre chain has.....links.

29. In surveying one should proceed from.....

30. If we work from part to whole the error will tend to..... .

31. ..... is the most important line in chain surveying.

32. Height of tower can be found by using chain, tape and..... .

Answers. (1) superior. (2) $100 (\sec\theta - 1)$ (3) stage (4) mm. (5) $\frac{1}{250}$ to $\frac{1}{1000}$ (6) less than 30° or more than 120°. (7) triangulation. (8) roof (9) 0.9×10^{-6} per 1°C. (10) chain, tape. (11) 250 sq. km. (12) horizontal plane, and vertical line (13) 64% steel, 36% nickel. (14) the agencies of Governments. (15) scale, larger (16) subtractive. (17) chain (18) 20cm (19) 45°. (20) to set right angles on the chain line. (21) Reciprocal or indirect ranging. (22) 5 links. (23) swivel (24) cumulative (25) reconnaissance. (26) sloping (27) chain. (28) 150 (29) whole to part. (30) increase. (31) base line (32) Ranging rod.

Choose the correct answer from the given alternatives in each of the following :

1. The main principle of surveying is to work from
   (a) part to the whole (b) whole to the part
   (c) lower to higher level (d) higher to lower level.

2. The methods of plane surveying can be used when the extent of area is less than
   (a) 500 sq. km (b) 5000 sq. km (c) 2500 sq. km (d) 250 sq. km.

3. Which is the follwing scale smallest one ?
   (a) 1cm = 10m (b) R. F. = $\frac{1}{5000}$ (c) 1:10,000 (d) 1cm = 10km.

4. The distance between two points A and B on a plan was found to be 100m

when measured with a scale with R. F. of $\frac{1}{1000}$. The distance between the same points when measured with a scale of R. F. = $\frac{1}{500}$ will be

(a) 200m (b) 50m (c) 2500m (d) 1000m

5. Assuming that a draftsman can plot a distance as small as 0.25mm, and the scale of plotting is 1 in 10,000, the distances in the field can be measured to the nearest
(a) 0.25m (b) 2.5m (c) 0.025m (d) 25.0m.

6. The difference between an arc distance of 18.5 km on the surface of the earth and the corresponding chord distance is less than
(a) 10mm (b) 100mm (c) 1mm (d) 1000mm

7. If the error makes the measurements too great, it is a
(a) positive error (b) Compensting error (c) negative error.

8. A good surveyor should always try to achieve
(a) Accuracy (b) Precision (c) Both accuracy and precision.

9. A surveyor forgets to count one chain length while measuring a line. The distance measured will have a
(a) Systematic error (b) Accidental error (c) Mistake.

10. The co-efficient of linear expansion for invar as compared to that of ordinary steel is about
(a) $\frac{1}{500}$ (b) $\frac{1}{100}$ (c) $\frac{1}{30}$ (d) $\frac{1}{200}$

11. If h is the difference in height between the end points of a chain of length L, the slope correction is approximately equal to

(a) $-\frac{h}{2L}$ (b) $-\frac{h^2}{2L}$ (c) $-\frac{2h^2}{L}$

12. If a 20m chain diverges a perpendicular distance of 2m from its correct alignment, the error in length is
(a) + 0.20 m (b) -0.20 m (c) + 0.10 m (d) - 0.10 m

13. An invar tape is made of an alooy of
(a) Nickel and steel (b) Brass and steel
(c) Copper and steel

14. A metallic tape is made of
(a) Metal (b) Metallic wires and cloth (d) Steel

15. The temperature correction of the tape
(a) is always negative (b) is always positive (c) may be positive or negative.

16. If the length of a chain line along a slope of θ degrees is L, the required slope correction is
(a) $+ 2 L \cos^2 \theta/_2$ (b) $- 2 L \cos^2 \theta/_2$
(c) $+ 2 L \sin^2 \theta/_2$ (d) $- 2 L \sin^2 \theta/_2$

17. For a well-conditioned triangle, no angle should be less than
(a) 45° (b) 30° (c) 15° (d) 60°

18. A tie line is run
(a) to check the accuracy of field work
(b) to locate details which are away from the chain line.
(c) between the main survey stations.
(d) parallel to the survey line.

19. The angle of intersections of the mirrors in an optical square is
(a) 60° (b) 45° (c) 30° (d) 90°

20. The allowable length of an offset depends upon the
(a) scale of plotting. (b) degree of accuracy required
(c) Both (a) and (b).

21. A French cross staff is used to set out the lines at an angle of
(a) 45° (b) 90°
(c) Both and (d) Neither (a) nor (b)

22. Chain surveying is generally used for
(a) small areas in open ground
(b) small areas which are crowded by features
(c) large areas in open ground
(d) large areas which are crowded by features

23. The permissible error in chaining on rough or hilly areas is usually
(a) 1 in 100 (b) 1 in 250 (c) 1 in 1000 (d) 1 in 10,000

24. The surbsidiary line is the same as
(a) range line (b) tie line (c) survey line (d) base line

25. An offset scale always has the zero mark atits
(a) middle. (b) end. (c) may have its middle or at the end.
(d) None of above.

26. The directions of survey lines, in chain surveying are fixed by
(i) angular measurements (ii) forming a network of triangles
(iii) prismatic compass (iv) forming a network of well-conditioned triangles

27. Object of surveying is
    (i) to measure the area of a plot of land
    (ii) to prepare map of the plot.
    (iii) to locate the details of features in the field.
    (iv) all the above.

28. In measuring a long distance between two points accurately the method adopted is
    (a) measuring directly with chain without other accessories.
    (b) measuring directly with metallic tape without other accessories.
    (c) with ranging and chaining method.
    (d) triangulation.

Answer : 1 (b); 2 (d); 3 (d); 4 (b); 5 (b); 6 (b); 7 (a); 8 (c); 9 (c); 10 (c); 11 (b); 12 (c); 13 (a); 14 (b); 15 (c); 16 (d); 17 (b); 18 (b); 19 (b); 20 (c); 21 (c); 22 (a); 23 (b); 24 (b); 25 (c); 26 (iv); 27 (ii); 28 (c);

DIAL SURVEYING

Fill in the blank spaces correctly

(1) Dial surveying is carried out when the extent of area is..... .

(2) Dial surveying is:..... than theodolite traversing.

(3) The least count of a prismatic compass is..... .

(4) With a primatic compass.....bearings can be measured.

(5) If the bearings of two lines AB and CB are S45°E and N45°W, the angle ABC equals to..... .

(6) If the true bearing of a line is 99°48' and declination 2°18', the magnetic bearing of the line is.....

(7) Reduced bearing is alwyas.....90°.

(8) The permissible error in prismatic compass is..... .

(9) The zero is marked at the south in the case of..... .

(10) The lines connecting points at which the needle points to the true north are known as..... .

(11) In tubular compass the direction of the..... is defined when the pointer is exactly midway between the two.....lines.

(12) The trough compass is used for obtaining.....quickly.

(13) The line joining the places having equal declination is known as..... .

(14) Magnetic equator is imaginary line along which dip is..... .

(15) In an underground roadway in a mine.....survey is preferred.

(16) In an underground roadway stations are fixed in.....for carrying out survey work.

(17) The magnetic dip at the magnetic pole is..... .

(18) Agonic lines pass through points of..... .

(19) The horizontal angle beween the true meridian and the magnetic meridian is called..... .

(20) The figures of graduations in prismatic compass are.....

(21) The positions of East and West points are..... on a Miner's Dial.

(22) The fore and back bearings of a line should differ by..... .

(23) .....traverses provide checks on the measured angles.

(24) The north and south magnetic poles are located approximately.....Km and.....Km, respectively, from the true geographical poles.

(25) The quadrant bearing is also called as.....bearing.

(26) If the fore bearing of AB is N35°E, the fore bearing of BA is..... .

(27) The bearing of any line in relations to.....is called true bearing and it is a..... quantity.

(28) The changes in the amount of.....is known as variation.

(29) Grid north lines are strictly..... .

(30) The.....method is most commonly used in mine surveying.

(31) Permissible errors in Miner's dial closed traverse are (a) linear error.....and (b) angular error.....

Answers : (1) large. (2) less accurate (3) 30'. (4) whole circle (5) 180° (6) 102°06' (7) less than (8) 10' $\sqrt{N}$ when N is number of sides of a closed traverse. (9) prismatic compass. (10) agonic line. (11) magnetic meridian, vertical (12) magnetic meridian (13) isogonic line (14) 0°. (15) dial (16) roof (17) 90°. (18) zero declination (19) declination (20) inverted (21) transposed (22) 180°. (23) closed (24) 1600, 2500 (25) reduced (26) S35°W. (27) Geographical or true meridian, constant (28) declination (29) parallel (30) double fore-sight (31) 1 in 1000, $(20 + x)\sqrt{n}$ sec. Where x is least count and n is the number of stations.

Choose the correct answer from the given alternatives in each of the following :

1. The angle of dip at the magnetic pole is
   (a) 0° (b) 90° (c) 45° (d) None of the above

2. Agonic lines pass through points
   (a) zero declination (b) equal declination
   (c) zero dip (d) equal dip

3. A negative declination shows that the magnetic meridian is due to the
   (a) West of the true meridian (b) East of the true meridian
   (c) South of the true meridian (d) None of the above

4. The horizontal angle between the true meridian and the magnetic meridian is called
(a) Dip (b) Azimuth (c) Declination (d) None of the above

5. The graduations in a prismatic compass
(a) are inverted (b) have zero at south
(c) are from 0° to 360° (d) all the above

6. The quadrant bearing of a line is determined by a
(a) Prismatic Compass (b) Miner's Dial
(c) Optical Square (d) None of the above

7. The vertical angle between the longitudinal axis of a freely suspended needle and the horizontal line is called
(a) Declination (b) Dip
(c) Azimuth (d) None of the above

8. At some points on earth's surface there are a series of places having same magnetic dip and a line joining such points in called
(a) Isogonic line (b) Agonic line
(c) Isoclinic line (d) Acclinic line

9. The magnetic bearing of a line is the horizontal angle between
(i) the line and the magnetic meridian,
(ii) the line and the geographical meridian,
(iii) the line and an arbitrary meridian,
(iv) the magnetic meridian and the geographical meridian.

10. The positive declintion shows that the magnetic meridian is due to the
(i) west of the true meridian, (ii) south of the true meridian,
(iii) east of the true meridian, (iv) north of the true meridian.

11. In the whole circle system of bearing, the value lies between
(i) 0° and 90°, (ii) 0° and 180°, (iii) 0° and 270°, (iv) 0° and 360°,

12. Lines, connecting points at which the needle points to the true north, are known as
(i) isogonic lines, (ii) agonic lines,
(iii) chain lines, (iv) survey lines.

13. When there are sources of local attraction in the vicinity the method of surveying by loose-needle dial is
(i) Reciprocal bearing method (ii) Ordinary loose-needle method
(iii) Back-and-foresight method (iv) Double-foresight method

Answers : 1 (b); 2 (a); 3 (a); 4 (c); 5 (d); 6 (b); 7 (b); 8 (c); 9 (i); 10 (iii); 11 (iv) 12 (ii); 13 (i)

PLANE TABLE SURVEYING

Fill in the blanks with suitable words

(1) Alidade is used for.....in plane table survey.

(2) The use of U-fork is..... .

(3) The use of field book is.....in plane table survey.

(4) The plane table is used for.....and.....simultaneously in the field.

(5) .....is less costly than theodolite survey.

(6) The principle of plane table surveying is based on.....and..... .

(7) The best method for solving three-point problem is.....method.

(8) In trial and error method for solving three-point problems.....rules are used.

(9) The strength of fix is.....when the station point is nearer to the great circle.

(10) Graphical method in plane table survey is named as..... .

(11) The working edge of the alidade is called.....edge.

(12) The trial and error method is also known as.....method.

(13) Correct orientation in plane tabling is done.....method.

(14) For locating the position of two inaccessible objects.....method of plane tabling is used.

(15) The instrument used for centering the plane table is..... :

(16) Plane tabling is not a.....method of surveying.

(17) Plane table survey is suitable in.....areas.

(18) When the point sought is inside the great triangle the.....will be inside the great triangle.

(19) In setting up a plane table, the operation which is done first is..... .

(20) The instrument which is used in plane table surveying for determining the horizontal distances without actual measurement is called..... .

(21) For the scale of plotting of 1 in 400, the permissible error in centring of plane table is about..... .

(22) Two-point and three-point problems are method of.....and..... .

Answer : (1) Sighting and drawing rays. (2) for transferring the ground point on to the sheet. (3) eliminated (4) surveying, plotting (5) plane tabling (6) symmetry, parallelism (7) trial and error (8) Lehman's (9) poor (10) bessel's graphical method. (11) fiducial (12) triangle of error (13) back sighting (14) intersection (15) plumbing fork (16) rapid (17) magnetic (18) triangle of error (19) centring. (20) telescopic alidade. (21) 0.1 m. (22) resection, orientation.

Choose the correct answer from the given alternatives in each of the following :

1. In setting up a plane table, the operation which is done first is
   (i) levelling, (ii) orientation, (iii) centering, (iv) resection.

2. The process of keeping the plane table into a fixed direction so that a line representing a direction on the plan is parallel to its direction on the ground is called
   (i) centering, (ii) orientation, (iii) levelling, (iv) Resection.

3. The instrument used for centering the plane table is
   (i) alidade, (ii) plumbing fork, (iii) trough compass, (iv) optical square.

4. Two-point and three-point problems are methods of
   (i) Traversing, (ii) Resection only,
   (iii) Resection and orientation, (iv) All the above.

5. Three-point problem can be solved by
   (i) Lehman's method, (ii) Bessel's method,
   (iii) Mechanical method, (iv) All the above.

6. The two-point problem as compared to the three-point problem is
   (i) more accurate, (ii) quicker,
   (iii) more laborious, (iv) all the above.

7. In plane table surveying, the trial and error method is also known as
   (i) Mechanical method (ii) Lehmann's method
   (iii) Tracing paper method (iv) Bessel's method

8. The method of plane tabling sometimes known as graphic triangulation is
   (i) Radiation (ii) Intersection
   (iii) Progression (iv) Resection

Answer : 1 (iii); 2 (i); 3 (ii); 4 (iii); 5 (iv); 6 (iii); 7 (ii); 8 (ii)

LEVELLING

Fill in the blanks with suitable words

(1) The curved surface which is parallel to the mean speroidal surface of the earth is .....surface.

(2) A.....is at all points equidistant from the centre of the earth.

(3) The levels of underground roadways and workings shall be related to.....,an arbitrary datum.....below the M.S.L.

(4) The point on which both the fore-sight and back-sight are taken is called a..... .

(5) Line of collimation is also known as..... .

(6) The apparent movement of image relative to cross-harirs of telescope is..... .

(7) The.....is a fixed point of known elevation above datum

(8) The two most common defects in the telescope are.....and..... .

(9) The most commonly used eye piece of a telescope is the.....eye piece.

(10) For levelling instruments, the magnification of the telescope is usually.....diameters.

(11) An.....combination is a compound lens which is free from the spherical aberration.

(12) An achromatic combination is a compound lens which is free from the..... .

(13) The unit of power of a lens is the..... .

(14) Most of the telescopes used in surveying instruments are of.....type.

(15) The Ramsden eye-piece magnifies the image, but the image remains..... .

(16) The erecting eye-piece has the disadvantage of loss of.....due to two..... .

(17) The.....of a spirit-level is its ability to exhibit small deviations from horizontal.

(18) The sensitivencess of a spirit-level is directly proportional to the..... .

(19) For very precise work and when the sights are long a target staff known as.....is used

(20) The automatic level is also known as.....level.

(21) The parallel plate micrometer is also known as..... .

(22) .....are performed at each set up of level prior to taking observations.

(23) Permanent adjustments of levels consist in establishing a fixed relationship between the .....of the levels.

(24) The collimation error of a dumpy level is tested and adjusted by.....method.

(25) The only condition of permanent adjustment of a tilting level is..... .

(26) The.....is the height of the line of sight of the telescope above datum.

(27) The correction for curvature is.....,the observed staff reading.

(28) The correction for refraction is.....that of curvature and.....to the observed staff-reading.

(29) The combined effect of curvature and refraction is to.....the staff readings, and combined correction is..... .

(30) The errors of curvature and refraction can only be eliminated by adopting.....levelling.

(31) Subsidence plans should be prepared at the.....scale at which.....plans are maintained. Scale for vertical section should be 1cm to..... .

(32) The depth of shaft may be measured by direct measurement using.....and by indirect measurement employing a..... .

(33) The.....levelling is suitable in steep gradients where dumpy level is inconvenient.

(34) The permissible closing error for ordinary levelling with a dumpy level is..... .

(35) The.....between any two consecutive contours is called the contour interval.

(36) The horizontal distance between two consecutive contours is known as the..... .

(37) Contour lines are.....when the gradient is uniformly sloping, and when the ground is plane they are..... .

(38) Contour lines close together near the.....and wide apart at the..... .

(39) Contour lines cross.....at right angles.

(40) The direction of the steepest slope at a point on contour is at.....to the contour.

(41) H.F.L. is not a.....line.

(42) .....*with a long spirit-level fitted at its top is used to maintain gradient approxi*-mately.

(43) A ground may be considered as level if the slope of the ground is..... .

(44) The.....tripod is commonly used in surveying work.

(45) In 4 metre levelling staff each metre is sub-divided into.....divisions.

(46) The thickness of each division in a staff is..... .

(47) Grade contour is located approximately by means of a.....

(48) .....method of contouring is very slow and tedious.

(49) Permissible errors in levelling
- (i) surface levelling.....
- (ii) underground levelling.....
- (iii) Precise levelling.....
- (iv) Trigonometric levelling.....
- (v) Straight-edge levelling.....
- (vi) Surface Bench Mark.....
- (vii) Shaft inset Bench Mark.....
- (viii) Inbye Bench Mark.....
- (ix) Subsidiary points.....

Answers : (1) level (2) level line (3) National Datum-Mines, 5000m (4) change point. (5) line of sight. (6) parallax. (7) Bench Mark (8) chromatic, spherical aberration (9) Ramsden (10) 25 to 40 (11) aplantic (12) chromatic aberration (13) diopter. (14) internal-focussing (15) inverted. (16) brilliancy, extra lenses. (17) sensitivity (18) radius of curvature. (19) Philadelphia rod (20) self-aligning (21) optical micrometer. (22) Temporary adjustments (23) fundamental lines (24) Two-peg (25) collimation adjustment. (26) height of collimation (27) subtractive from (28) one-seventh, additive (29) increase, subtractive. (30) reciprocal (31) same, statutory, 1m. (32) steel band or tape, plumb wire. (33) trigonometrical (34) $\pm$ 24 k mm where k is the distance in km. (35) vertical distance (36) horizontal equivalent. (37) equally spaced, straight and parallel. (38) top of the hill, flat ground. (39) ridge lines (40) right angles (41) contour (42) wooden template (43) 0^0 (44) telescopic (45) 200 (46) 5mm (47) clinometer (48) Indirect

(49)
- (i) ..... 2cm per km
- (ii) ..... 4cm per km
- (iii) ..... 1mm per km
- (iv) ..... 40cm per km
- (v) ..... 1in 100 to 1 in 1500
- (vi) ..... $\pm$ 2cm per km
- (vii) ..... $^1/_{5000}$ th of depth
- (viii) ..... $^1/_{2500}$ of inclined length
- (ix) ..... $\pm$ 50 cms.

Choose the correct answer

1. An imaginary line lyingh throughout the ground surface and having a constant inclination to the horizontal is a
   (i) contour line (ii) contour gradient
   (iii) ridge line (iv) valley line

2. The imaginary straight line which joins the optical centre of the object-glass with the point of intersection of the cross-hairs of the diaphragm is
   (i) level line (ii) horizontal line
   (iii) line of collimation (iv) None of the above

3. Height of instrument in levelling work in the same set of instrument position is determined by
   (i) R.L of staff placed at B.S. position + B.S. reading.
   (ii) R.L of staff placed at F.S. position + F.S. reading.
   (iii) R.L of staff placed at I.S. position + I.S. reading.
   (iv) all the above

4. The arrangement of a levelling tube is
   (i) Diaphragm - eye-piece - objective
   (ii) Eye-piece - objective - diaphragm
   (iii) Objective - diaphragm - eye-piece

5. Radius of the earth is nearly
   (i) 25000 km (ii) 6000 km
   (iii) 4000 km

6. The correct calculation is
   (i) RL = HI + FS (ii) HI = FS - RL
   (iii) HI - FS = RL (iv) FS = RL + HI

7. The depth of shaft may be measured by
   (i) Indirect measurement by EDM
   (ii) Indirect measurement by plumb wire
   (iii) Indirect measurement by steel band or special shaft tape.

8. In steep gradients where the dumpy level is inconvenient, the method of finding altitudes is known as
   (i) Reciprical levelling (ii) Precise levelling
   (iii) Differential levelling (iv) Trigonometrical levelling

9. The direct method of contouring is
   (i) a quick method (ii) an accurate method
   (iii) used for very large areas (iv) none of the above

10. The contour interval is
    (i) directly proportional to the scale of the map.
    (ii) inversely proportional to the scale of the map.
    (iii) large for flat ground.
    (iv) small for hilly areas.

11. In reciprocal levelling, the error which is not fully eliminated is due to
    (i) earth's curvature (ii) inclined line of sight
    (iii) refraction (iv) none of the above

12. The constant vertical distance between two adjacent contours is called
    (i) contour gradient (ii) contour interval
    (iii) horizontal equivalent (iv) none of the above

13. The angle between the vertical and the fault plane is known as
    (i) want (ii) throw
    (iii) heave (iv) hade.

14. When the ends of broken strata overlap and a vertical boring proves the same seam twice the fault is known as
    (i) transcurrent fault (ii) normal fault
    (iii) reverse fault (iv) none of the above.

Answer : 1 (ii); 2 (iii); 3 (iv); 4 (iii); 5 (ii); 6 (iii); 7 (ii); 8 (iv); 9 (ii); 10 (ii); 11 (iii); 12 (ii); 13 (iv); 14 (iii)